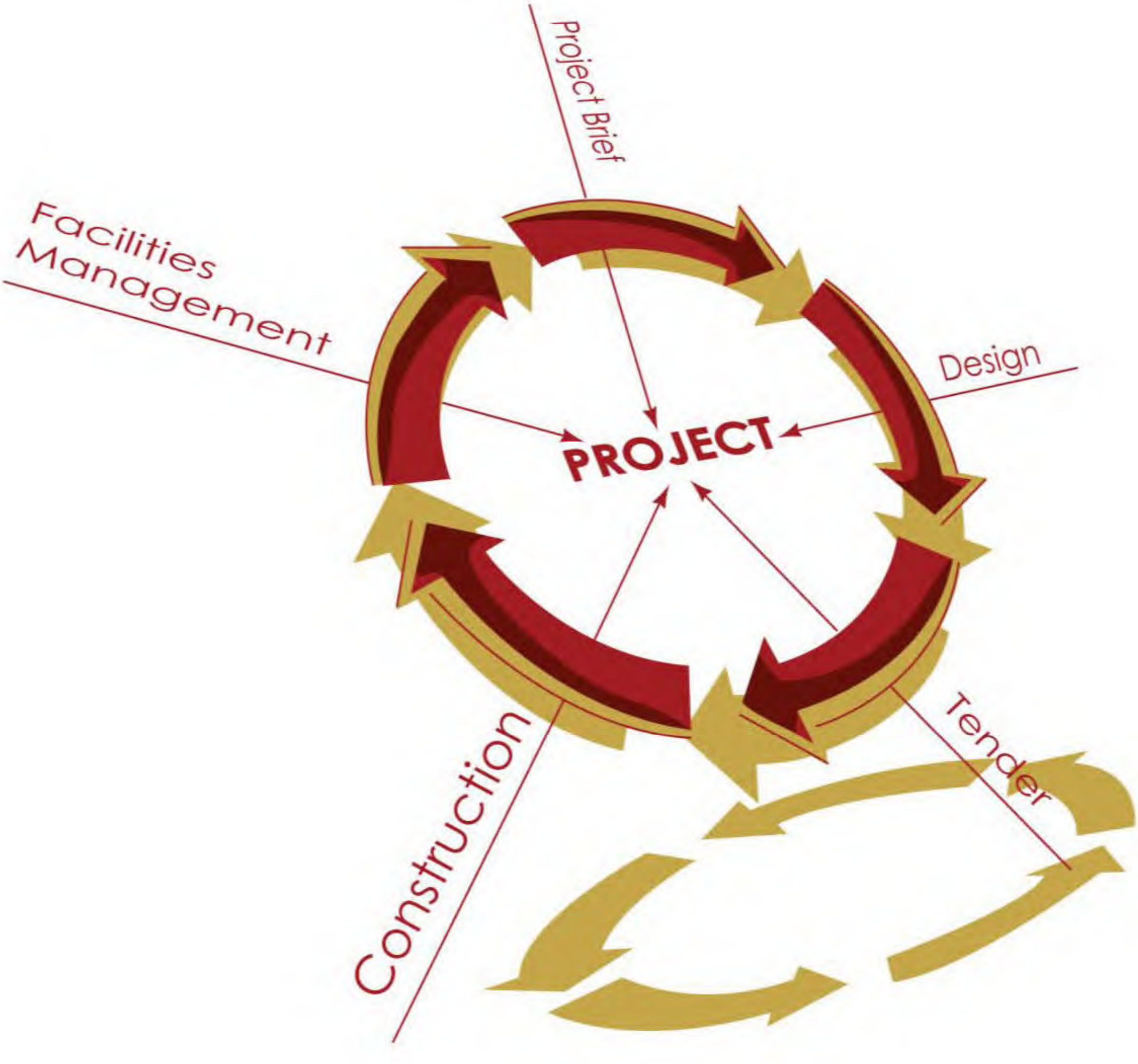
MCRJ Volume 6 | No. 1 | 2010

ISSN 1985-3807

Malaysian Construction Research Journal



CREAM
CONSTRUCTION RESEARCH INSTITUTE OF MALAYSIA

CIDB MALAYSIA

Contents

Editorial Advisory Board

Editorial

Welcome from the Editors

Welcome to the sixth issue of Malaysian Construction Research Journal (MCRJ). In this issue, we are pleased to include another six interesting papers from various contributors that cover the wide range of research area in construction industry. The editorial team would like to express our sincere gratitude to all contributing authors and reviewers for their contributions, continuous support and comments. It is hope that this issue will bring beneficial information to the readers.

Hanizam Awang and Wan Hamidon study the flexural performance of roof panel constructed from profiled steel sheeting connected to dry board. The system called profiled steel sheeting dry board (PSSDB) is proposed as an alternative to the conventional forms of roof construction. Using finite element modeling (FEM), they analyze the performance of the system such as bending, elastic behavior and deflection as well as the effect of their thickness on the deflection.

Nor Hayati Abdul Hamid presents the seismic assessments on precast wall panels using fragility curves. Three types of precast wall were constructed i.e. monolithic reinforced concrete walls, slender precast reinforced concrete wall with slenderness ratio of 60 and prestressed precast hollow core wall. Experimental works were carried out and the behavior of the wall such as cracks propagation, crack patterns, spalling, buckling and bar fractures were observed. Color coded system is used to identify damages states, performance level and ductility factors, while fragility curve is used to assess their seismic performance using classification damage states.

Mahyuddin Ramli and Eethar Thanon Dawood investigates the performance of high strength flowing concrete containing steel fiber. This paper highlights the use of varying percentage of steel fibre as volumetric fractions on the high strength flowing concrete (HSFC) to determine the mechanical properties, which include density, compressive strength, flexural strength, splitting tensile and static modulus of elasticity. Results show that the increase percentage of steel fiber will improve the strength and mechanical properties of HSFC.

Ahmad Baharuddin Abd. Rahman, ***et al.*** study the performance of grouted sleeve connectors subjected to incremental tensile load. The paper investigates the behavior and the strength performance of grouts sleeve connectors for joining recast concrete components. These connectors were subjected to incremental tensile loads until failure. Their performances were evaluated in terms of stiffness, yield strength, ductility and failure modes.

Mastura Jaafar, *et al.* present the assessment of women involvement in the construction industry. Based on survey questionnaires sent to women contractors in Northern Malaysia, it is revealed that the majority of women entrepreneurs started with small business, which for some successfully grew. Most of them gained business knowledge through formal courses and received Government assistance. The authors concluded that it is not impossible for women to achieve success in this male dominated industry.

The final paper by **Ting Sim Nee, *et al.*** analyse the payment affected by standard forms of construction contract in Sarawak. The purpose is to create better understanding of the explicit clauses in various standard forms relating to payment. Analysis was made of three different standard forms in East Malaysia, i.e. the Malaysian Public Works Department 75 (PWD 75), the CIDB Standard Form of Contract for Building Works 2000 edition (CIDB 2000) and the Malaysian Standard Form of Contract 1998 edition (PAM 1998). Findings show that the prevalent standard form in Sarawak fails to clarify various issues such as penultimate claims, account preparation procedures, time frames for settlement and submission of final claim. The findings show these problems are prevalent in the industry and changes and improvements are crucial to bring betterment to the industry and all the contracting parties.

Editorial Committee

FINITE ELEMENT PREDICTION ON THE FLEXURAL PERFORMANCE OF INDIVIDUAL PANEL OF PROFILED STEEL SHEET DRY BOARD (PSSDB) ROOF SYSTEM

Hanizam Awang[1], Wan Hamidon Wan Badaruzzaman[2]
[1]School of Housing, Building and Planning, Universiti Sains Malaysia, 11800, Penang, Malaysia
[2]Department of Civil and Structural Engineering, Universiti Kebangsaan Malaysia, Selangor, 43600 Malaysia.

Abstract
Roof panel system constructed from profiled steel sheeting connected to dry board (known as profiled steel sheeting dry board (PSSDB)) by self –drilling, self tapping screws is being proposed as an alternative to the traditional forms of roof construction. In the proposed system, the normal position of the PSSDB system has been reversed. The board plays an important role in providing the flat surface for the roofing system and also to enhance the strength of the panel system. This kind of structure has a significant advantage of removing the internal trussing and support that is normally required in a traditional trussed roof system, hence creating additional useable space within the roof space, which is advantageous to homeowners. This paper looks into the possibility of employing the PSSDB panel to form a complete roof structure, analyzing it under the effect of bending test using the already established and verified finite element method (FEM). The basic behaviour in the elastic range is studied to check for deflection as the main controlling design factor. The effect of the steel sheeting thickness on the deflection of the mid span panel is investigated. In addition, the effect of using timber strip along the edge side of the roof panel as the method of enhancing the structure is also studied. The finite element model developed has shown accuracy within 5% to 11% compared to experimental results in predicating the deflection of the PSSDB panel. It was found that, the stiffness of the panels increased with the thickness of the steel profiled and the use of timber connecting strips along the side between adjacent PSSDB panels. Thicker panel and the present of timber strip along the edge side of the panel have increased the span of the panel for the loading value considered.

Keywords: *Profiled steel sheet, dry board, timber strip, deflection*

INTRODUCTION

The composite systems (also called mixed or hybrid systems) have seen widespread use in decades because of the benefits of combining the two construction materials. In order to meet the above requirement the composite system known as profiled steel sheeting dry board (PSSDB) system was proposed. The PSSDB system, a thin-walled, lightweight composite structure consisting of profiled steel sheet connected to dry boards by self drilling, self tapping connectors, is a structural load bearing system described in earlier publication by Wright et al. (1989). The connectors play an important role in transferring horizontal shear between the boarding and the profiled steel sheeting while the board plays dual role, firstly providing a flat surface for roofing and secondly, enhancing the stiffness and strength of the system through composite action. Figure 1 shows the typical composition of PSSDB system.

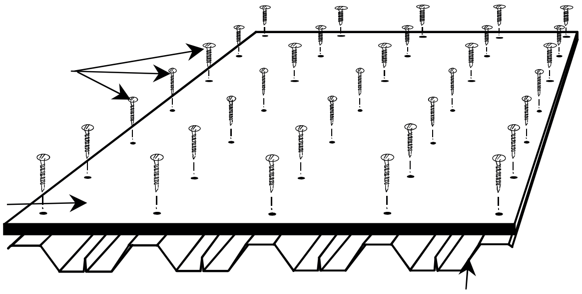


Figure 1. Typical PSSDB system

Originally, the idea of using PSSDB system as a structural component was first introduced by Wright and Evans (1986) as a replacement to existing timber joist floor in domestic building construction. The idea was then extended and studied in-depth by researchers utilizing locally available materials in Malaysia. The structural performance of the panels using local materials has been shown to be acceptable and can be deployed successfully in composite PSSDB floor panels. Some previous study of the behaviour of PSSDB system was reported by Ahmed (1996), Ahmed et al. (2000), Ahmed and Wan Badaruzzaman (2003, 2005), Akhand (2001), Benayoune and Wan Badaruzzaman (2000), Wan Badaruzzaman et al. (1996), Wan Badaruzzaman et al. (2003) and Wan Badaruzzaman and Wright (1998). The studies include structural and non-structural performance of the system. The application of the PSSDB system is not only limited to floor and wall application but also to roof system.

The potential of assembling PSSDB panel to form folded plate roof structures was first studied by Wan Badaruzzaman (1994). He has developed an analytical solution and a computer program to predict the behaviour of orthotropic folded plate structures simply supported on two end diaphragms. The program was successfully verified using full-scale mansard type roof structure (Figure 2). However the proposed analytical approach can only be used to solve simple structures.

Figure 2. PSSDB folded roof structure tested by Wan Badaruzzaman

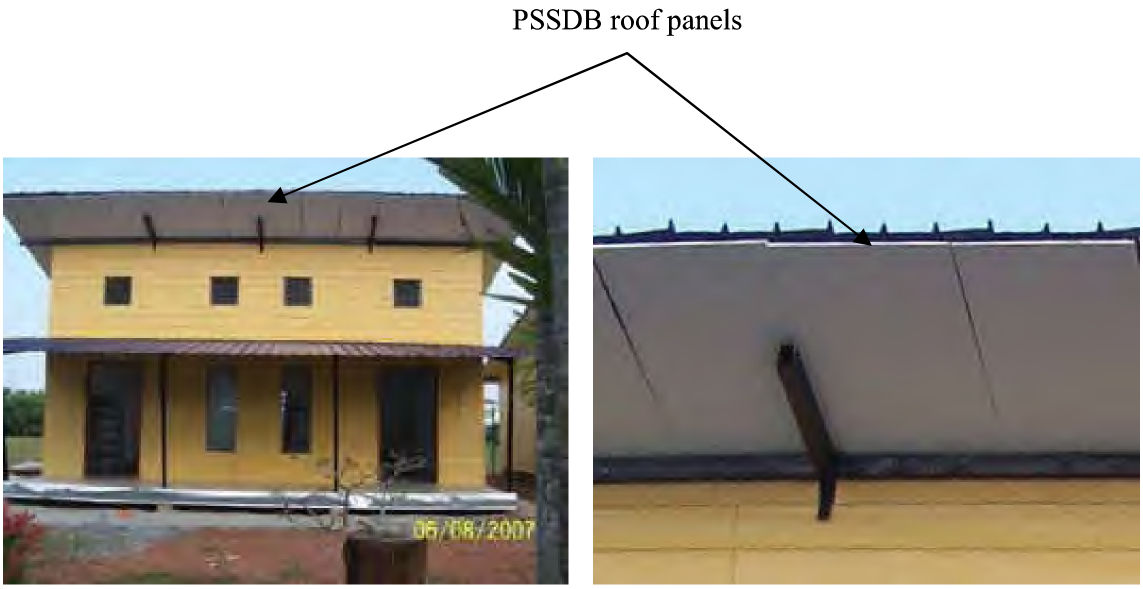
Ahmed (1999) extended the study on a simple pitch folded plate roof. An analysis based on finite element was used and the prediction gave an acceptable degree of accuracy. A study by Ahmed et al. (2000) using folded plate models showed that the structural behaviour of PSSDB system was satisfactory to be used in practice. The analysis was performed by means of finite element modeling in three dimensional and two directional plate elements acting as a shear connector to allow partial interaction between components. Ahmed et al. (2003) also studied the theoretical and experimental aspects of the PSSDB folded plate structure. The used of fixed end support conditions reduces the mid-span central ridge deflection up to 23.7% compared to simple support. Figure 3 shows the model developed by Ahmed (1999).

Figure 3. PSSDB folded roof structure tested by Ahmed (1999)

There are many advantages of the PSSDB roof system when compared to traditional forms of pitched roof structures in small and medium sized buildings which normally would involve the use of either purlins and rafters or a trussed rafter system. One clear advantage is due to the load bearing capacity of the PSSDB system and another is due to the more durable materials it is made of. Some of the advantages can be simplified as below (Awang and Wan Badaruzzaman, 2007):

i. The structure of the roof that would normally involve a considerable number of internal elements that would impinge on the roof space and nullifies its effective use would no more be required.
ii. Considerable numbers of connections between elements normally required in the skeletal framing which are often difficult to form and add to the cost, would be eliminated.
iii. The difficulty to provide the overall stability of the roof structure which involves cross bracing and an allowance for wind uplift would now be removed.
iv. Insect attack and rotting of roof timbers; a problem that is not always resolved with preservatives and treatments would no more be a threat.

Based on the original concept of PSSDB system and the studies on the folded plate PSSDB roof structures, a new concept of roof system has been developed. This paper reports on the finite element modeling (FEA) technique to predict the structural behaviour of individual PSSDB roof panel system. Such a practical application of roof panel system can be seen in Figure 4.



(a) Full view of PSSDB roof panel (b) Closed up view

Figure 4. The PSSDB roof panels

THE PROPOSED INDIVIDUAL PSSDB PANEL

The PSSDB roof panel was constructed using Ajiya Cliplock CL 660 (profiled steel sheet) and Prima*flex* (dry board). The Prima*flex* is an autoclaved cellulose fiber reinforced cement flat board. It will not deteriorate when exposed to sun, rain, wind, dampness and dryness (HUME, 2007). The board was not designed to provide high tensile resistant and to carry any loads other than self-weight.

The thicknesses of the sheeting and the board were 0.48 mm and 9 mm respectively. The sheeting and dry board were screwed together using self-tapping and self-drilling screws at a distance of 100 mm on every rib of Ajiya CL 660. The screws of type DX-RW (4.2 mm diameter and 25 mm long) produced by Powerdrive (1991) was used. The normal position of the PSSDB panel is shown in Figure 1. However, the roofing panel in the normal PSSDB position could pose durability problems in the long run, as the dry board is exposed to the weather.

In order to solve this problem, the position of the PSSDB panel was reversed (see Figure 5). The new position of the board will provide for a flat surface on the underside of the roof facing into the room. This flat surface will eliminate the use of suspended ceiling panel in buildings.

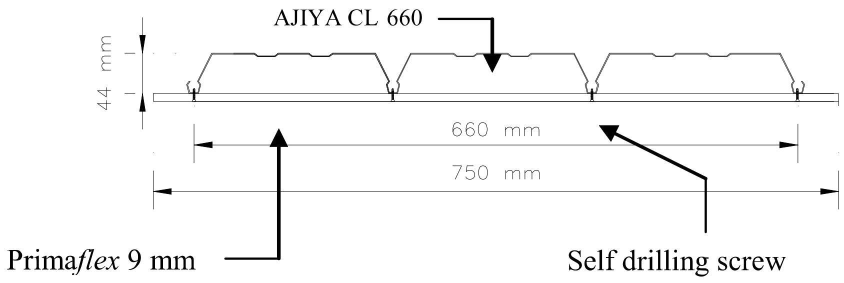


Figure 5. Cross section Sample 1

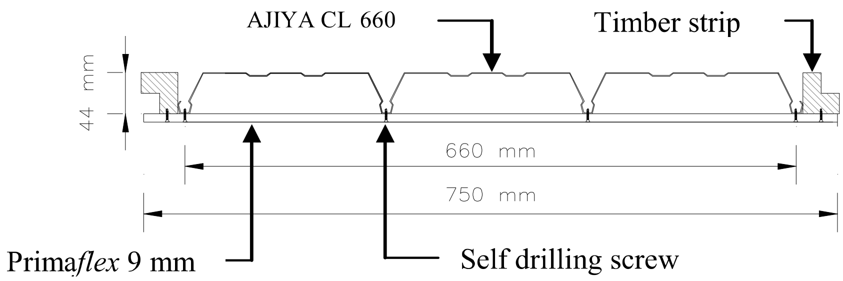


Figure 6. Cross section Sample 2

FINITE ELEMENT ANALYSIS

The model was developed into two stages. Model Sample 1 was a roof panel without timber strip along the longitudinal span, whereas Model Sample 2 was enhanced by attaching a timber strip by means of self drilling, self tapping screws. The test panel length was 2000 mm. The theoretical analysis was based on FEA using LUSAS finite element software (LUSAS, 2003). The finite element modelling technique was validated for experimental PSSDB panel as reported in an earlier publication (Akhand, 2001, Sodiq, 2004, Awang, 2008, Awang and Badaruzzaman, 2009). Each component of PSSDB panel such as profiled steel sheeting, dry board, timber strip, screw connection between profiled steel sheeting to dry board and screw connection between dry board to timber strip are modelled according to the already established technique, with some improvement, and would be mentioned briefly in this paper.

Modelling of profiled steel sheet

The profiled steel sheeting was modelled using isotropic thin shell elements, QSL8. The QSL8 is a thin, doubly curved isoparametric element formed by applying Kirchoff constrains to a three-dimensional degenerated thick shell element. The formulation of this element takes into account both membrane and flexural deformations. However as required by thin shell theory, transverse shearing deformations are excluded.

Modelling of board

The Prima*flex* was also modelled using isotropic thin shell elements. The thin shell elements, QSL8 as used in modelling the profiled steel sheeting were also used in modelling the dry board. The input materials properties were the Young's modulus and Poisson's ratio values.

Modelling of timber strip

The timber strip along the longitudinal span was modelled using solid hexahedral elements, HX20. The HX20 is a family of 3D isoparametric solid continuum elements with higher order models capable of modelling curved boundaries. The elements are numerically integrated. The Young's modulus and Poison's ratio of timber are 7000 MPa and 0.2 respectively.

Modelling of connection

The discrete screwed connections between the profiled steel sheet and dry board were modelled as spring elements which have a combination of translational and rotational deformation. The connections were modelled as close as possible to represent the action of shear connection. Compatible joint spring element, JL43 was chosen for this purpose. In numerical model, the joints were used to connect the board and sheeting at discrete screw locations only. Therefore, 'dummy' spring elements, JL43, were used to model the contact plane between profiled steel sheet and dry board to avoid them from behaving independently of each other. The 'dummy' joint spring elements were used to connect all the nodes at intermediate locations. The characteristic of the 'dummy' joint elements were the same as an active joint element which represented the screw connections except that it had very small spring stiffness along the contact plane.

The FE model and boundary condition

Figure 7 and 8 show the finite element idealization of the model structures Model Sample 1 and Model Sample 2 respectively. The input properties such as the geometry and dimensions of the components, the Young's modulus and Poisson's ratio of the individual materials, and connection modulus were derived from either the manufacturers' details or determined experimentally. The model takes advantage of the symmetrical nature of the roofing system, where only half of the roof structure was modelled employing appropriate boundary conditions at the symmetrical axis and end supports.

The panels were analysed for a uniformly distributed load for 5 increments (0.25 to 1.25 kN/m^2). The model structure was simply supported (translational movement in vertical and transverse direction were restrained) at the ends on supporting frame. For symmetrical loading situation, only one quarter of the structure panel was modeled.

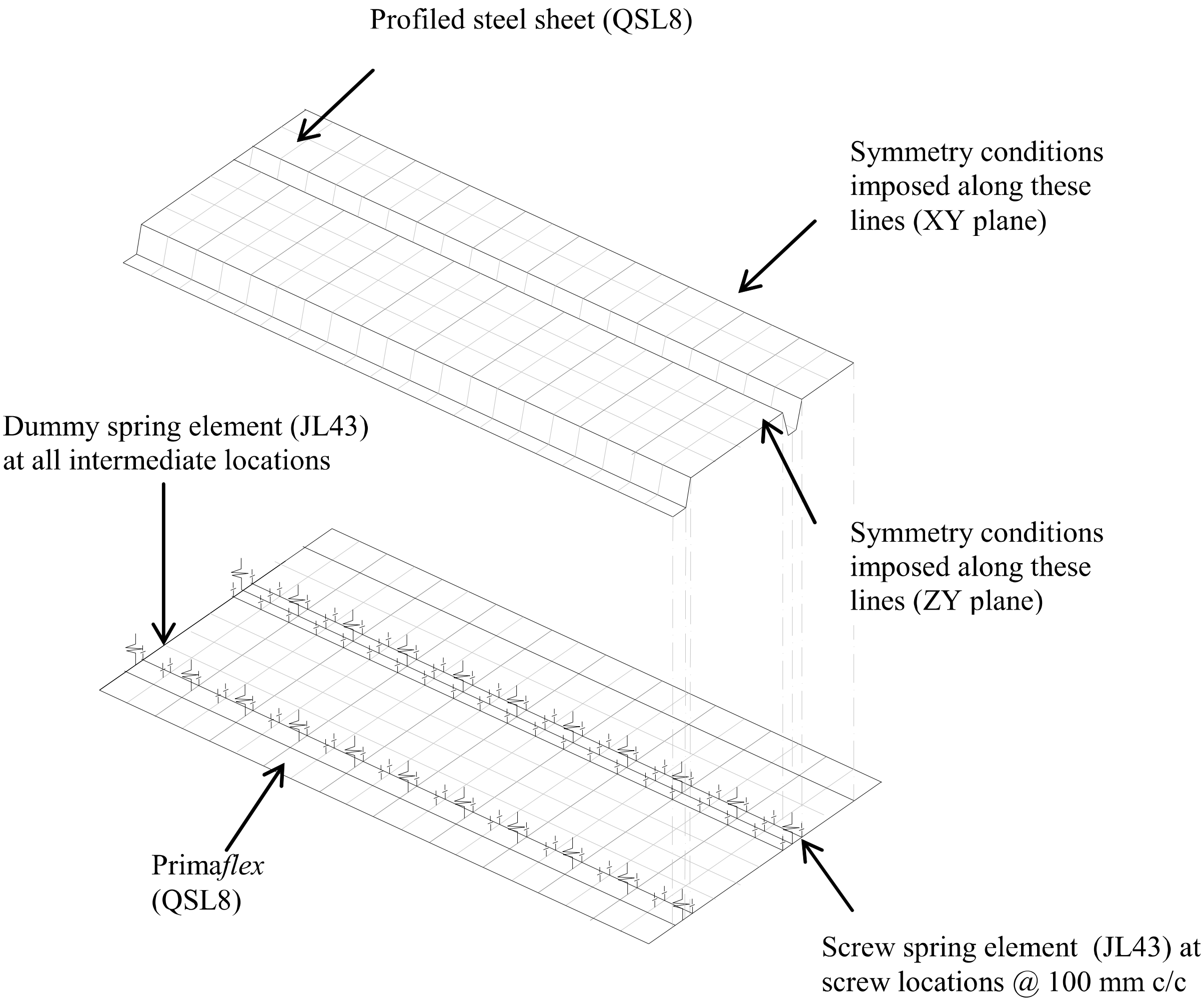


Figure 7. Finite element idealisation of panel without timber strips, Sample 1

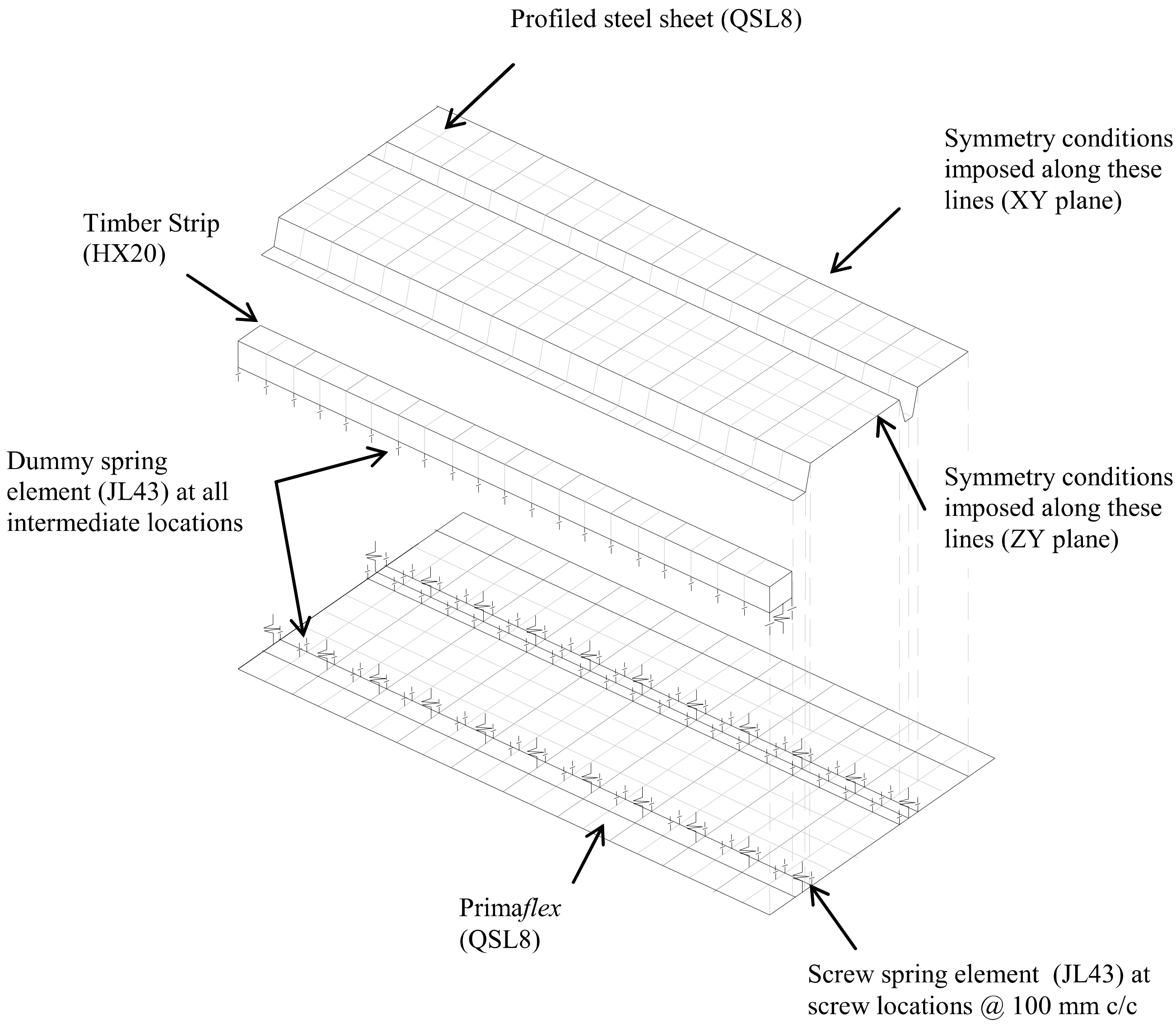


Figure 8. Finite element idealisation of panel with timber strips, Sample 2

VALIDATION OF FINITE ELEMENTS MODELLING

In order to validate the proposed linear model, the models described above were constructed and tested in the laboratory. The test programme consisted of two series of full scale tests. The models were tested on a simple span using a whiffle tree loading to simulate a uniformly distributed load. The load was applied through four steel loading beams to the samples. The deflection values were measured using displacement transducers. The transducers were located at the middle and quarter span along the mid span line. The transducers were also located at both ends of the mid width line to detect any unintentional unsymmetrical eccentricity of loading.

Sample 1 and 2 then were modeled and analysed. Figure 9 shows the load - deflection curve for the Finite Elements results plotted together with the test data. It can be seen that, the deflection values (at mid span mid width) obtained from FEM follow closely the experiment results (within the linear elastic range). Reasonable accuracy was observed with discrepancies varied from 5.6% to 11.2%. Therefore, the finite element prediction with some safety factors can be used to design the PSSDB panel system at serviceability stage without any loss of accuracy or being too conservative.

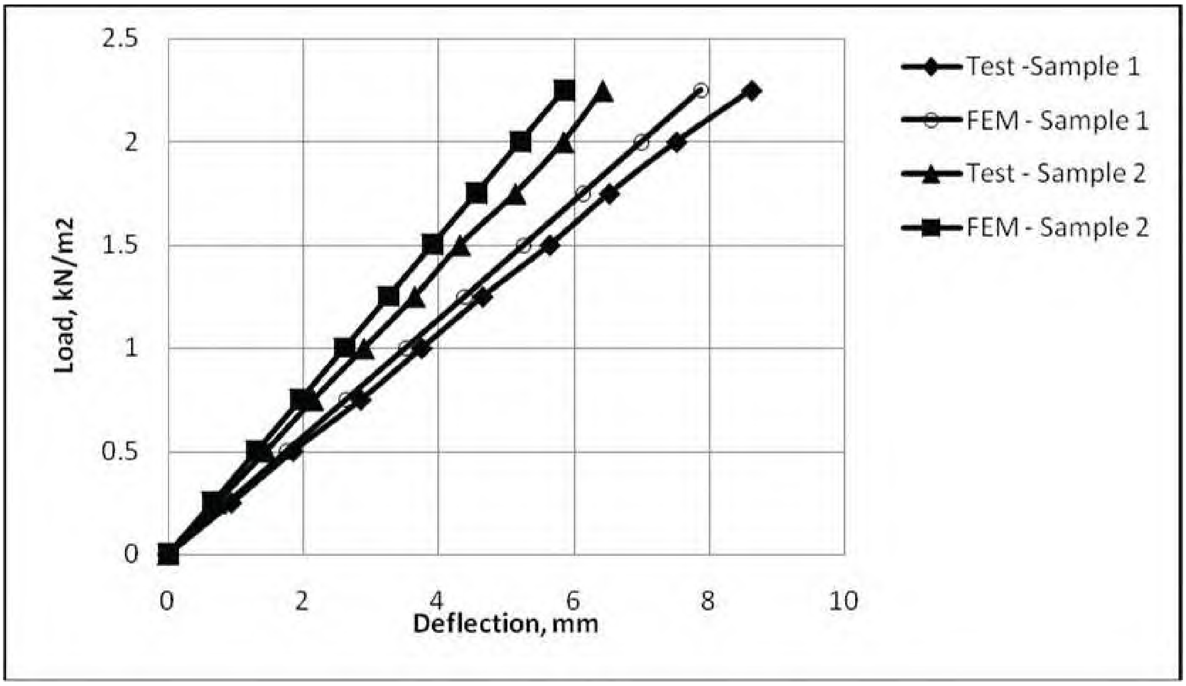


Figure 9. Load deflection curve for the Sampel 1 and 2

RESULT AND DISCUSSION ON FEM PARAMETRIC STUDIES

The parametric studies included the effect of the thickness of profiled steel sheet and the effect of using timber strip along the longitudinal edge of panel. For the study of the effect of profiled steel sheet thickness, the board, Prima*flex* was fixed as 9 mm, whilst thicknesses of the profiled were varied from 0.48 mm to 1.0 mm. For the effect of timber strip, both the board and the sheeting were fixed.

Effect of profiled thickness on mid span deflection

For the loading value of 0.75 kN/m^2, as the thickness of profiled steel sheeting increased from 0.48 mm to 1 mm, the midspan deflection of every case decreased from 35 % to 41 %. The analysis results show that, for the 0.48 mm thickness, the maximum span allowed is limited to 3m. This is to ensure that the deflection due to service load will not exceed the deflection limit of *l*/200. For stiffer panels, having the thickness 0.55 mm to 0.88 mm, the maximum span allowed is 3.5 m while for the 1 mm thick, the panel can be put on service up to 4 m long. It can be concluded that, the thicker profiled steel sheeting will increase the stiffness of the panel. Load not exceeded the deflection limit of *l*/200 were deemed as significant for design purposes (shown in the Table 1).

Table 1. Deflection values due to different thickness of profiled

Thickness (mm)	Span (mm) 2000	2500	3000	3500	4000
0.48	2.63	5.21	11.77	19.5	31.03
0.55	2.38	4.78	10.41	17.58	28.26
0.60	2.24	4.53	9.68	16.51	26.68
0.80	1.82	3.77	7.76	13.55	22.19
1.00	1.54	3.06	6.60	11.68	19.27

Effect of the timber strip on mid span deflection

The values of midspan deflection and the corresponding loads are given in Tables 2 and 3. Results from the analysis show that, all the deflection values at panel 2 m and 2.5 m for the two cases were within the range of serviceability state of design for the loading value considered. The values of midspan deflections at panel 3 m and above depend on the loading considered. The span of both panels, with and without timber strip with 0.25 kN/m^2 loading, can reach up to 4 m length. The increase of load to 0.5kN/m^2, has limited the length of panel in Sample 1 (without timber) to 3 m length. However, the length of panel span in Sample 2 (with timber strip) can reach up to 4 m length. From the results, the panel with timber jointing seems to perform relatively better than panel without timber strip. The use of timber strip in PSSDB panel will increase the spanning length of panel.

Table 2. Deflection at midspan (Sample 1- without timber strip)

Load kN/m^2	Span (mm) 2000	2500	3000	3500	4000
0.25	0.88	1.74	3.92	6.5	10.34
0.50	1.75	3.47	7.84	13.0	20.68
0.75	2.63	5.21	11.77	19.5	31.03
1.0	3.51	6.94	15.69	26.01	41.37
1.25	4.39	8.68	19.61	32.51	51.71

Table 3. Deflection at midspan (Sample 2-with timber strip)

Load kN/m^2	Span (mm) 2000	2500	3000	3500	4000
0.25	0.78	1.58	3.32	5.40	8.91
0.50	1.58	3.14	6.65	10.8	17.80
0.75	2.38	4.72	9.98	16.21	26.72
1.0	3.18	6.32	13.31	21.61	35.62
1.25	3.98	7.90	16.63	27.10	44.53

CONCLUSION

This paper has described the possibility of employing the PSSDB panel to form a complete roof structure, analyzing it under the effect of bending test using the already established and verified finite element (FEM) technique. The basic behaviour in the elastic range is studied to check for deflection as the main controlling design factor. In addition, the effect of using timber strip along the edge side of the roof panel as a method of enhancing the structure is also studied. By comparison of the midspan deflection in elastic region, it was found that the finite element method has shown accuracy within 5% to 11% compared to experimental results in predicting the deflection of the PSSDB panel. From the results, the sample with timber jointing strip (Sample 2) seems to perform relatively better than sample without timber strip (Sample 1). The presence of timber strip along the edge side of the panel has made it possible for the increase in span of the panel for the loading value considered. It can be concluded that the timber strip plays an important role in stiffening the roof panel system. The PSSDB panel with reverse position has been shown in this paper to be potentially useful as a load bearing roof panel.

REFERENCES

Ahmed, E. (1999). Behaviour of Profiled Steel Sheet Dry Board Folded Plate Structure. PhD. Thesis. Universiti Kebangsaan Malaysia.

Ahmed, E. and Wan Badaruzzaman, W.H. (2003). Finite Element Prediction of the Behavior of Profiled Steel Sheet Dry Board Folded Plate Structures – An Improved Model, *International Journal of Engineering, Transaction B: Applications*, 16(1), pp. 21-32.

Ahmed, E. and Wan Badaruzzaman, W.H. (2005). Finite Element Prediction on the Applicability of Profiled Steel Sheet Dry Board Structural Composite System as a Disaster Relief Shelter, *Journal of Construction and Building Materials*, 19(4), pp. 285-295.

Ahmed, E., Wan Badaruzzaman, W.H. & Rashid, A.K. (1996). A Simplified Elastic Composite Floor Section Analysis with Incomplete Interaction. *Jurnal Kejuruteraan UKM* **8**: 67-78.

Ahmed, E., Wan Badaruzzaman, W.H. & Wright, H.D. (2000). Finite Element Elastic Analysis of Profiles Steel Sheeting Dry Board Single Span Composite Panels. *Proceeding of ASCCS Conference* (6) : 1083-1090.

Akhand, A.M. (2001). Nonlinear Finite Element Modeling and Partial Plastic Analysis of Composite Profiled Steel Sheeting Dry Board Continuous Floor. PhD. Thesis. Universiti Kebangsaan Malaysia.

Awang, H., 2008. Development of Profiled Steel Sheeting Dry Board (PSSDB) Roof Panel System. PhD. Thesis. Universiti Kebangsaan Malaysia.

Awang, H. and Wan Badaruzzaman, W.H. (2009). The Bending Behavior of a Reversed Profiled Steel Sheet Dry Board (PSSDB) Panel for Application in a Roofing System, *Journal of Construction in Developing Countries*. Vol.14, No. 2, pp 21- 36.

Awang, H. and Wan Badaruzzaman, W.H. (2007). Development of Profiled Steel Sheet Dry Board (PSSDB) Roofing System. *Proceeding of International Conference on Built Environment*, Penang.

Benayoune, A.G. & Wan Badaruzzaman, W.H. (2000). Limit State Behaviour of Profiled Steel Sheeting/Dry Board Wall Panel. *Proceeding of APSEC 2000*, pg. 39-46.

Hume Cemboard Bhd. (2007). *Primaflex : General purpose cladding sheet*. Petaling Jaya.

Powerdrive Sdn. Bhd. (1991). *Fastening system* (Risalah Powerdrive Self-drillers), Malaysia.

LUSAS. (2003). *LUSAS Finite Element System*. FEA Ltd., United Kingdom.

Sodiq, H.(2004) Performance of New Profiled Steel Sheeting Dry Board Floor System With Concrete Infill. PhD. Thesis. Universiti Kebangsaan Malaysia.

Wan Badaruzzaman, W.H. (1994), The Behaviour of Profiled Sheet/Dry board System. Ph.D Thesis. University of Wales.

Wan Badaruzzaman, W.H., Ahmed, E. & Rashid A.K. (1996). Out-of Plane Bending Stiffness Along the Major Axis of Profiled Steel Sheet Dry Board Composite Floor Panels. *Jurnal Kejuruteraan Awam* **8** : 79-95.

Wan Badaruzzaman, W.H. & Wright, H.D. (1998). Lightweight Thin Walled Profiled Steel Sheeting/Dry Board (PSSDB) Composite Flooring System. *Proceeding of International Conference on Thin-Walled Structures* (2) : 355-365.

Wan Badaruzzaman, W.H., Zain, M.F.M, Akhand, A.M. & Ahmed, E. (2003). Dry Boards as Load Bearing Element in the Profiled Steel Sheet Dry Board Floor Panel System-Structural Performance and Applications. *Journal of Construction and Buildings Materials* **17** : 289-297.

Wright, H.D. & Evans, H.R. (1986). Profiled Steel Sheeting for the Replacement of Timber Flooring in Building Renovation. SERC Grant GR/D/76875. United Kingdom.

Wright, H.D., Evans, H.R. & Burt, C.A. (1989). Profiled Steel Sheet/Dry Boarding Composite Floors. Struct. Engineering **67** (7/4): 114-129.

ASSESS SEISMIC PERFORMANCE OF THREE TYPES OF CONCRETE STRUCTURAL WALL PANELS USING FRAGILITY CURVES

Nor Hayati Abdul Hamid[1]
[1]Senior Lecturer, PhD, Institute of Infrastructure and Sustainable Management, Faculty of Civil Engineering, University Teknologi MARA, 40450 Shah Alam, Selangor, Malaysia,

Abstract
Seismic assessments on three types of precast wall panels using fragility curves are presented herein. These walls are monolithic reinforced concrete walls which designed in accordance to NZ3101 (New Zealand Standard Code of Practice), slender precast reinforced concrete wall with slenderness ratio (Height/thickness of the wall) of 60 and prestressed precast hollow core wall which designed using Damage Avoidance Design (DAD) philosophy. Their seismic performances were observed during experimental work and then, the classification of their damage states are accordance to drift limits. Visual observations during experimental work were recorded such as crack propagations, crack patterns, spalling and crushing of concrete, buckling and fractures of longitudinal bars before classify them into damage-states limit. Damage states of these walls panels are followed by their definitions and descriptions are given in HAZUS. Colour-coded system is used to identify damages states, level of performances and ductility factors, while fragility curve is used to assess their seismic performance using classification damage-states. The level of safety for these wall panels in reinforced concrete buildings are assessed using fragility curves and the prediction of damage states are based on Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE).

Keywords: *Colour-coded System, Damage States Limit, Fragility Curves, Design Basis Earthquake, Maximum Considered Earthquake*

INTRODUCTION

When a major earthquake strikes the metropolitan areas, one of the challenging jobs is to evaluate the level of safety in RC buildings. Visual inspections by structural engineers are based on cracks identification, buildings/structures deformation and structures' stability. The structural engineer expertise's needs to decide whether the buildings require for demolishing or repairing depending on the amount of damages. They need to classify the damage states based on visual observation obtained from site. Pictures, descriptions and evaluation of structural damage contain in the reports are highly important to estimate direct and indirect economic losses and the period for retrofitting the structures. In order to obtain a good relationship between the site damages and total assessment of the structural performance, fragility curve is used to estimate global behaviour of structures starting from elastic behaviour until collapse state. Therefore, this study focuses on the seismic assessment for three types of wall panels together with level of safety under DBE (Design Basic Earthquake) and MCE (Maximum Considered Earthquake). These walls are conventional monolithic wall designed according to current standard NZ 3101 (1995), slender precast wall panels which violated the slenderness ratio exceeding 30 and precast hollow core panels designed using Damage Avoidance Design philosophy.

FINDING FROM PREVIOUS RESEARCHERS

Relationship between peak ground acceleration (PGA) and structural damage is frequently used to estimate the distribution of structural damages in buildings over certain seismic regions. Blejwas and Bresler (1978) proposed damage states of structures can be measured by taking the ratio of demand on the seismic response over the capacity of the system. Meanwhile, Banon et al.(1981) defined damage state parameters in terms of rotation ductility, curvature ductility, flexural damage ratio (FDR) and normalized cumulative rotation (NCR). Later on, Banon and Veneziano (1982) pointed out the necessity to define the terms flexural damage ratio (FDR) and normalized cumulative ratio (NCR). They defined the flexural damage function (FDR) as the ratio of initial flexural stiffness to the reduced secant stiffness and normalized cumulative rotation (NCR) is the ratio of cumulative plastic rotations in n_{cycle} cycles to the yielding rotation of the nonlinear spring. However, Park and Ang (1985) expressed the seismic damage of reinforced concrete structures as a linear combination of maximum deformation and absorbed hysteretic energy. To prove this relationship, extensive damage analyses of Single Degree of Freedom (SDF) system and a typical Multi-Degree of Freedom (MDF) reinforced concrete building were performed. Theoretical results showed a simple relationship between the destructiveness of seismic ground motion in terms of characteristic intensity and structural damage in terms of Damage Index (DI).

Further study was conducted by DisPasquale and Cakmak (1990) on global damage indices for the complex structures using an optimal time variant linear model fitted to strong motion records. They discovered a good correlation between the numerical values of damage indices with actual visual observation of the structures. More explorations on building damage functions made by Kircher et al. (1997) for earthquake loss estimation using others parameters such as ground shaking characteristics, site/soil amplification and shaking durations. Further verification was made by O'Rourke and So (2000) on the seismic performance of 400 water tanks in nine separate earthquake events. They realized that the relative amounts of stored water contents in the tank and the tank's height to water tank diameter ratio had a significant influence on the tank's seismic performance.

After defining the parameters on structural damages, structural indices and loss estimation, another method is required to assess the probability of damages states in relation to ground motion. This method is known as fragility curve. Fragility curve can predict the probability of reaching or exceeding specific damage states for a given level of peak earthquake response. The probability of being in a particular state of damage and the input used to predict building-related losses are calculated by taking the difference of damage states in the fragility curve analysis. The expected seismic performance of the structures system can be achieved by combination the fragility curves, probability of ground shaking and an integrated possible outcome such as Monte-Carlo simulation (Singhal and Kiremidjan, 1996). One application of fragility curves was tested on gravity-type quay walls (Ichii, 2004). He proposed design charts based on effective stress-based FEM and some parametric study on gravity-type quay walls. Then, fragility curves were generated by considering the difference between the observed displacements in case histories and estimated displacement by the chart. These proposed fragility curves are useful in assessing the restoration cost of the wall after an earthquake, real-time damage level and optimization of cost-benefit analysis under the requirement of seismic performance level. Up to date, there is no assessment on precast wall using fragility curve. Therefore, this study focuses on the seismic assessment for three different types of wall panels together with level of safety under DBE and MCE. These walls are conventional monolithic wall designed according to current standard NZ 3101, slender precast wall panels which violated the slenderness ratio exceeding 30 and precast hollow core panels designed using damage avoidance design philosophy.

CHARACTERIZATION OF DAMAGES FOR THREE TYPES OF WALL PANELS

There are several models which can be used to quantify the damages, characterization of damage state and estimation of losses after the earthquakes. One of the models used in this research is called "HAZUS 99-SR2" which had been developed by Federal Emergency Management Agency (FEMA) and National Institute of Building Science (NIBS). The primary objective of HAZUS 99-SR2 (2004) is to provide a methodology and software application to develop earthquake losses on a regional scale. The loss estimation is useful for local, state and regional officials to plan and stimulate efforts in reducing risks from earthquakes and to prepare for emergency response or recovery. The status for damage states for the overall buildings after an earthquake is tabulated in Table 1. The damage state of overall performance buildings which made from these types of wall panels will be stated using Table 1 which based on collected evidences and expected ductility factor from sites. Detail descriptions of the performance level for buildings have been clearly defined by ATC (1995) and FEMA (1997). These codes of practices defined four standard performance levels together with their description as shown in Table 2. These two tables are used to define damage states and standard performance for buildings which made from three types of wall panels. For validation of their damages states, three models of a single precast wall panel were designed, constructed and tested under quasi-static lateral cyclic loadings in heavy structural laboratory. Visual observation of structural damage together with its level of drifts are recorded and measured. After that state and drift damage are quantified using the colour-coded, numerical format, performance level and ductility factor.

Table 1. Definition of Damage States (HAZUS 99-SR2, 2004)

Damage State	HAZUS Descriptor	Post earthquake Utility of Structures	Evidence	Outage time	Expected Ductility Factor
1	None	No damage	None (pre-yield)	-	1
2	Slight	Slight damage	Cracking	< 3 days	2
3	Moderate	Repairable damage	Large cracks cover spalled	< 3 weeks	3
4	Heavy	Irreparable damage	Failure of components	< 3 months	4
5	Complete	Irreparable damage	Partial/total Collapse	> 3 months	6

Table 2. Definition of Standard Performance Levels (ATC, 1995 and FEMA, 1997)

Level Performance	Descriptions
Fully Operational	Only minor damage, buildings retain their original stiffness and strength, nonstructural components operate and the building can be used, and the risk to life is very low.
Functional	Only minor damage, structures retain nearly all their original stiffness and strength, nonstructural components are secured and utilities are functional, repairs may be instituted at the convenience of the building users, and risk of life is low.
Life Safety	Significant structural and nonstructural damage, the building has lost a significant amount of its original stiffness, but retains some lateral strength and margin against collapse, nonstructural components are secure but not operating, the buildings may not be safe to occupy until repaired.
Near Collapse	A limiting damage state in which substantial damage has occurred, the building has lost most of its original stiffness and strength, and has little margin against collapse. Nonstructural components may be dislodged and present a falling hazard.

The first specimen of fixed based monolithic wall panel is designed according to NZ3101 (1995), constructed and tested on strong floor under quasi-static lateral cyclic loading. Figure 1 shows the overall seismic performance and hysteresis loops of a conventional reinforced concrete wall at 1.0% and 3.0% drift which was tested by Holden (2001). Figure 1(a) shows the cracks on the north face of the wall at 0.5% drift but it still maintained its stiffness and strength. This type of building remained fully functional and the facility continues in operation at this drift level. Figure 1(b) shows more and wider opening cracks at bottom corner of the walls and the buildings are still operational. Figure 1(c) depicts spalling and crushing concrete cover at both bottom corners of the wall and buckling of longitudinal bars at 2.0% drift. This type of buildings which designed using fixed-base monolithic wall panels have severe damages, decreasing its stiffness and the buildings should be evacuated. Figure 1(d) exhibits the fracture of longitudinal bars with massive crushing of concrete at 3.0% drift. This building is not safe to live and partially collapsed due to losing its stability when the concrete crumbled and longitudinal reinforcement bars were buckled. Figure 1(e) shows the hysteresis loops at 1% drift where the wall panel is already exceeding the yielding strength and displacement and behave in nonlinear regions. Figure 1(f) shows the hysteresis loops of the wall panel at 3.0% drift where the wall lost its strength (strength degradation) and become partially collapsed. After measuring the ductility factor and identify drift damage for this type of wall panel, the colour-coding which comprises green, yellow, orange and red are tagging against performance level, description level, drift damage and ductility factor as shown in Table 3. The combination of Table 1, Table 2, Table 3, visual observations and experimental results from monolithic wall panel are used for developing fragility curves.

Table 3. Definition of colour-coding and performance level of monolithic wall panel

Tag Colour	Performance Level	Description of damage level	Drift Damage	Ductility Factor
Green	Operational	Minor cracks, no damage, warehouse occupiable	0.5%	1.0
Yellow	Functional	Wider cracks, initial spalling at corner of walls with moderate level of damage. The warehouses can be entered to remove belongings.	1.0%	1.5
Orange	Life Safety	Extensive spalling along bottom walls, longitudinal bars buckling with heavy damage on the walls. Warehouse can be entered for short periods for removing important items.	2.0%	3.0
Red	Near Collapse	Fracturing of longitudinal bars, no stability of structures, near collapse. The building cannot be entered.	3.0%	4.5

The second specimen of slender wall panel with slenderness ratio of 60 is designed in accordance to NZ3101 (1995), constructed and tested under lateral cyclic loading up 2.0% drift. Figure 2 shows the visual observation damages and hysteresis loops at 1.5% drift of slender/thin precast wall panel for in-plane and out-of-plane directions (2004). The slender wall experienced severe cracks on the north face at 1.0% drift and initial cracking at the western corner of the wall as shown in Figure 2(a). As the level of drift increased up to 1.5%, spalling and crushing of concrete occurred on both bottom corners of the wall as shown in Figure 2(b). This building which comprises of slender wall experienced moderate damages and need to be repaired for about three weeks before it can be occupied again. The risk of life-safety is very low and the owners of buildings are allowed to occupy them after some repairs have been made. Figure 2(c) illustrates the failure of the wall when the outermost longitudinal bars fractured and shifted 30mm away from their original place at 2.0%. The wall lost its strength and twisted at the centre of the walls as shown in Figure 2(d). The wall failed by lateral-torsional buckling, became unstable and had little margin to collapse. The hysteresis loops of the slender wall panel are shown in Figure 2(e) at 1.5% drift for in-plane directions and Figure 2(f) at 1.5% drift for out-of-plane direction. The colour-coding as shown in Table 4 is tagging against the description damage, drift damage and ductility factor as defined in Table 1 and 2 for slender wall panels (Holden, 2004).

Table 4. Definition of colour-coding for slender precast wall panel (Holden, 2004)

Tag Colour	Description of damage level	Drift Damage	Ductility Factor
Green	Minor cracks, no damage, warehouse occupiable	0.25%	1
Yellow	Wider cracks, initial spalling at corner of walls with moderate level of damage. The warehouses can be entered to remove belongings.	0.5%	2
Orange	Extensive spalling along bottom walls, longitudinal bars buckling with heavy damage on the walls. Warehouse can be entered for short periods for removing important items.	1.0%	3
Red	Fracturing of longitudinal bars, no stability of structures, near collapse. The building cannot be entered.	2.0%	4

Due to serious fracture of longitudinal bars and instability problem occurring in conventional wall and buckling of slender/thin precast wall panels, the third specimen which consists of prestressed precast hollowcore wall panel is designed in accordance to Damage Avoidance Design philosophy, constructed and tested under quasi-static lateral cyclic loading. Figure 3 shows the visual observation and hysteresis loops at 3.0% drift. Figure 3(a) shows the overall seismic performance of precast hollow core wall panel which tested at 1.0% drift. Figure 3(b) shows the seismic behaviour of wall panel at 2.0% drift where there is no structural damage occurred under lateral cyclic loading. A single precast hollow core wall tested on a shaking table demonstrated that there is no visible damage occurred starting from 0.1% drift up to 4.0% drift. Both bottom corners of the precast hollow core walls performed very well under biaxial loading at 2.0% drift as illustrated in Figure 4(c) and (d), respectively. The flag-shaped of the hysteresis loops obtained from experimental work showed that there is no residual displacement occurred and the wall back to its original positions without any structural damages.

The uplift of the precast hollow core wall did not fracture bars because the unbonded post-tensioned tendons and energy dissipators surrounding the bottom of the precast wall protected it from the impact and was capable of transmitting the axial and biaxial lateral force to the base plate. The hysteresis loops for 1.0% drift and 2.0% drift of this type of wall are illustrated in Figure 3(e) and (f), respectively. The overall seismic performance of the precast hollow core wall performed extremely well as compared to fixed-based monolithic reinforced concrete wall panels and slender precast wall panel. Table 5 shows the colour-coding is tagging against the descriptions damage level, drift level and ductility factor for precast hollow core wall panel.

Table 5. Definition of colour-coding using precast hollow core wall system

Tag Colour	Description of damage level	Drift level	Ductility Factor
Green	Pre-uplift of walls, no cosmetic damage, warehouse occupiable	0.5%	1
Yellow	The external fuses are yielding, no damage, warehouse occupiable	1.5%	3
Orange	Rocking response with minor cracks at the corners, no structural damage, P-delta effect for taller walls, and loss of prestressing of tendons/fuses. The building can be entered to remove belongings.	2.0%	4
Red	Fractures of tendons or external fuses, no clamping forces, Warehouse might collapse. Buildings cannot be entered.	4.0%	8

THEORETICAL DEVELOPMENT OF FRAGILITY CURVES

A fragility curve describes the probability of reaching or exceeding a damage state at a specified ground motion level. Thus, a fragility curve for a particular damage state is obtained by computing the conditional probabilities of reaching or exceeding that damage state at various levels of ground motion. The probabilistic hazard levels frequently used in FEMA (1997) and their corresponding mean return periods are tabulated in Table 6. By referring to Table 6, there are two main objectives of the probability in designing buildings using three types of wall panels under performance levels which are life safety and collapse prevention requirement. Under life safety requirements, the probability of occurring earthquake within 50 years is 10% and the return period is 500 years. In this study, life safety requirement for these buildings is designed under Basic Design Earthquake (DBE) for Wellington, New Zealand is taken as $F_vS_1 = 0.4g$ where g is defined as peak ground acceleration. The collapse prevention is requirement is defined as 2% probability occurrence earthquake exceeding 50 years with mean return period of 2500 years. The limit peak ground acceleration for collapse prevention or also known as "Maximum Considered Earthquake" (MCE) is taken as the value of $F_vS_1 = 0.8g$ for Wellington, New Zealand. However, the value for DBE and MCE is depending on the location of buildings closed to the earthquake epicentre. The DBE and MCE which denoted as dotted line in fragility curves can be used to predict the percentage of performance level such as operational, immediate occupancy, life safety and collapse prevention.

Table 6. Probabilistic Hazard Levels

Performance Level	Earthquake Having Probability of Exceedance	Mean Return Period (years)
Operational	50%/50year	75
Immediate Occupancy	20%/50year	225
Life Safety	10%/50year	500
Collapse Prevention	2%/50year	2500

In order to plot fragility curves for three different types of wall panel by including the design earthquake of DBE and MCE, the theoretical equations need to derive first. The first step of developing fragility curve is to set the spectral acceleration amplitude of an earthquake for a period of T = 1 sec and then, the drift damage limit must be converted into spectral acceleration units (A). Base shear demand (C_d) for period of the structures for high damping is given in equation (1) as stated by FEMA (1997):

$$C_d = \frac{SA}{TB_L} \tag{1}$$

in which S is soil type factor, A is the peak ground acceleration (normalized with respect to g), T is the period of vibration and B_L is the factor of damping which is taken as more than 5%. The second step is to calculate the structural period of vibration according to yield strength and displacement for Single Degree of Freedom (SDOF) as given in equation (2).

$$T = 2\pi\sqrt{\frac{m}{K}} = 2\pi\sqrt{\frac{W\Delta}{Fg}} = 2\pi\sqrt{\frac{\Delta}{C_c g}} \tag{2}$$

Where the base shear capacity of the structure is defined as $C_c = \frac{F}{W}$, where F is the yield strength (base shear) of the structure, W is seismic weight of the structures, Δ is the yield displacement of the structure, and K is the stiffness of the structures. By substituting equation (2) into equation (1) and equating base shear capacity equal to base shear demand, the equation becomes:

$$C_c^2 = C_d^2 = \frac{C_c g}{4\pi^2\Delta}\left(\frac{SA}{B_L}\right)^2 \tag{3}$$

Then, by substituting $\Delta = \theta H$, then $(SA)_i = 2\pi B_L\sqrt{\frac{C_c\theta H}{g}}$ (4)

The third step is to convert the damage drift limit to spectral acceleration. Equation (4) is used to convert from damage drift limit to the spectral acceleration in developing the fragility curves. The fourth step is to transform the spectral acceleration into cumulative probability function (CPF). According to Mander (2003), the items which should be considered in developing fragility curves by taking into account the theoretical cumulative probabilistic functions are as follows:

(i) the expected site-specific response characteristics;
(ii) the inelastic strength and deformation capacity of the structure;
(iii) damage limit states;
(iv) randomness of ground motion response spectral demand;
(v) uncertainties in modelling structural capacity.

The intersection of the capacity curve and appropriate damped elastic demand curve provides a “performance point” based on the estimate of the structural strength and displacement demand. The probability distributions over these two curves indicated the uncertainty and randomness of the structures performance with a wide range of possible performance outcomes. The randomness and uncertainty can be represented as probability distribution function. This distribution function can be expressed as a lognormal cumulative probability density function known as “fragility curve”. The cumulative probability function is give by equation (5) as

$$F(Sa) = \Phi\left[\frac{1}{\beta_{C/D}} \ln\left(\frac{S_a}{A_i}\right)\right] \tag{5}$$

where Φ = standard log-normal cumulative distribution function; S_a = the spectral amplitude (for a period of $T = 1\sec$); A_i = the median spectral acceleration necessary to cause the ith damage state to occur and $\beta_{C/D}$ = normalized composite log-normal standard deviation which incorporates aspects of uncertainty and randomness for both capacity and demand.

The fifth step is to use central limit theorem by incorporating the normalized composite log-normal standard deviation. The central limit theorem requires the composite performance outcome to be distributed log-normally. By using the derivation of this theorem, the coefficient of variation for lognormal distribution is given by Kennedy at el. (1980);

$$\beta_{C/D} = \sqrt{\beta_C^2 + \beta_D^2 + \beta_U^2} \tag{6}$$

The value of $\beta_C = 0.2$ represented as randomness of the structural capacity based on the analysis carried out by Dutta and Mander (2003). β_U = uncertainty associated with strength reduction factor and the global modeling process, the assumed values is ranging between 0.2 and 0.4. The overall value of $\beta_{C/D}$ is calibrated by Pekcan (1999) and validated by Dutta and Mander (1998) against fragility analysis based on the site data obtained in the 1994 Northridge Earthquake and the 1989 Loma Prieta Earthquakes which recommended the value to be $\beta_{C/D}$ = 0.6. After obtaining all the parameters of the cumulative probability function, the final step is to plot fragility curves for three different types of walls panels using lognormal distribution function as illustrated in the following section.

PLOTTING FRAGILITY CURVES

The hysteresis loops and visual observation of damages to a single precast wall panel for assessing the seismic performance of wall system is insufficient. The assessment of a single precast wall alone is not enough in determining the global damage state of the building and seismic performance of multi-panel walls systems. After obtaining the drift damage limit from three experimental works of wall panels which had been conducted in the laboratory, the fragility curve can be plotted based by using equation (5). Fragility curves of these wall panels were developed herein are used to quantify confidence level of occurrence of a certain damage state is either achieved or exceeded as a log-normal cumulative distribution function of peak ground acceleration. The percentage of a confidence interval is taken as one hundred percentages minus the percentage probability of failure. As the probability of failure becomes higher, the level of confidence towards seismic performance of structure is reduced or vice versa.

Figure 4 shows the fragility curves for three different types of single precast wall panels' performance when classified under coloured-coded and damage states numbering format. These fragility curves plotted based on equation (5) and equation (6) as derived above. The x-axis represents the Peak Ground Acceleration (PGA) which denoted as F_vS_1

and both y-axes represent Cumulative Probability Function (CPF) and Confident Interval which measured in percentages. The percentage Confident Interval (CI) is taken as the value of one subtracted from the value of Cumulative Probability Function and multiplied by 100%. On each graph of wall panel, the Design Basis Earthquake (DBE) equal to $F_v S_1 = 0.4g$ and Maximum Considered Earthquake (MCE) equal to $F_v S_1 = 0.8g$ for the case of Wellington, New Zealand. Wellington, New Zealand is classified as high seismic region and located on the Ring of Pacific Fire. The fragility curves for three types of walls are compared under DBE and MCE. Some correlation between life safety and functional limit can be obtained based on the plotted graphs and their interpretations under MCE and DBE are as follows.

Under MCE, 14% of the wall made from conventional reinforced concrete would be expected to be red tag or have collapsed as shown in Figure 4(a), 72% of slender precast wall panels would be red tag or have collapsed as depicted in Figure 4(c), and only 2% of the structures made from precast hollow core wall following damage avoidance design philosophy would be expected to be red tag or have collapsed as illustrated in Figure 4(e). Wall panels made from conventional reinforced concrete would have 30% slight damage, 44% repairable damage, 16% irreparable damage, 6% heavy damage and 4% collapse (Figure 4(b)).The worst scenario would have occurred on slender precast wall panels where there would be 45% irreparable damage, 40% heavy damage and 7% would be expected to collapse (Figure 4(d)). Figure 4(f) shows that the buildings made from precast hollow core walls will survive under MCE with only 8% repairable but most of the walls only have cosmetic damage and no structural damage.

Under DBE, the walls constructed according to the standard code of practice would be 3% red tag or have collapsed as shown in Figure 4(a) as compared to slender precast wall panels with 30% red tag or collapsed (Figure 4(c)). But the buildings which are constructed using precast hollow core wall panels will survive after the earthquake and remain functional without structural and nonstructural damage. Most of the precast hollow core walls in the building behaved very well without any visible damage when they uplifted and rocked in their own position as shown in Figure 4(e) and (f). Conventional reinforced concrete walls would expect to have 71% slightly damaged, 24% repairable, 3% irreparable and 2% heavy damage as shown in Figure 4(b). Slender precast wall panels would be expected to have 20% slightly damaged, 18% repairable, 52% irreparable, 10% heavy damage or near collapse as demonstrated in Figure 4(d).

Based on fragility curves interpretation, precast hollow core walls which designed in accordance to Damage Avoidance Design philosophy will survive under both earthquakes conditions (DBE and MCE), followed by conventional reinforced concrete wall panels and finally, slender precast wall panels connected to base foundation.

DISCUSSION

The experimental results based on three different types of wall panels were obtained and their results were used to develop fragility curves. Based on the analysis and experimental results, precast hollow core wall has the most excellent seismic performance under in-plane quasi-static cyclic loadings as compared to other types of walls. From experimental results, precast hollow core wall did not suffer any structural damages especially at bottom of wall panel as compared to monolithic wall panel which failed at bottom of wall panel and slender wall panel failed by lateral torsional buckling and spalling concrete cover. Furthermore, hysteresis loops of precast hollow core wall show the flag-shape behaviour where there is no residual displacement when the wall goes back to its original position whereas the hysteresis loops for slender walls and conventional monolithic wall panels indicate that there are quite bigger residual displacements which up to 60mm when reached the ultimate strength. Moreover, precast hollow core wall has self-centring mechanism where the post-tensioned tendons which located at centre of the wall behave within elastic limit.

Based on the analysis of fragility curves for these three different types of walls, the results show that only precast hollow core wall can survive under Design Basic Earthquake (DBE) and Maximum Considered Earthquake (MCE). It can be seen that under these conditions of earthquakes, the buildings which construct using precast hollow core wall are still in green colour coding which means that the buildings still under operational with minor cracks and no structural damage. The worst case occurs for slender precast wall panel where before reaching DBE and MCE, the buildings experience very severe damages such as extensive spalling of concrete, buckling of longitudinal bars and collapse of the wall panels. It is also classified with orange colour-coded and some of the structures cannot be repaired. Therefore, it can be conclude that the buildings which constructed using precast hollow core panels and using Damage Avoidance Design philosophy can fulfil the primary objective of building code provisions for earthquake which are to avoid the collapse of the buildings and it is safe under both conditions of earthquakes (DBE and MCE).

Finally, fragility curves which obtained from experimental results can be used as the basis for estimation of economic loss is either directly or indirectly if strong/extreme earthquakes strike certain areas in seismic regions. The government and private agencies can take some actions and precautions if there is a big earthquake strikes any areas especially located closed to the fault lines. It is also beneficial to the insurance companies to estimate the properties losses, business losses and economic losses after an earthquake. The owners of the buildings need to insured their properties under earthquake conditions even though it is the act of God. In order to reduce the risk of losing the properties and business, the owners of the buildings should buy insurances which cover the losses due to natural disaster such as earthquakes, flood, typhoons, landslides and sink holes.

CONCLUSIONS AND RECOMMENDATIONS

The conclusions of this study can be drawn as follows:

1) Extensive comparisons on visual observations and hysteresis loops were made based on three experimental results of wall panels which are fixed-base conventional wall, slender walls and precast hollow core walls. Precast hollow core wall shows the most excellent seismic performances in terms of visual observation, strength, stiffness, no residual displacement and has the self-centring mechanism as compared to the others two types of wall panels.
2) Colour coding for each type of wall indicates whether the buildings/warehouses are really safe to occupy or not. Damage drift limit which obtained from experimental

results and sites can be used to quantify the colour coding of the buildings which can be display at entrance of the buildings following the earthquakes.

3) The proposed fragility curve in this study is very useful for owner of the buildings, insurances companies, goverment and private agencies. Fragility curve can be used for the assessment of restoration cost after the earthquake and real-time damage evaluation level.
4) Damage Avoidance Design is recommended for the construction of warehouse building using precast hollow core walls especially in seismic regions. This design also can be proposed in construction of beam-column joint and bridge-piers connection.

REFERENCES

Applied Technology Council. (1995). Guidelines and Commentary for the Seismic Rehabilitation of Buildings. ATC-40, Redwood City, California.

Blejwas, B and Bresler, B., (1978). Damageability in Existing Buildings. EERC Rep. No.UCB 71-13, University of California, Berkeley, 1971 UCLA, March 1978, pp.505-512.

Banon, H., Biggs, J. M., and Irvine, H. M. (1981). Seismic Damage in Reinforced Concrete Frames. *Journal of Structural Division,ASCE*, 107(9), pp 1713-1729.

Banon, H., Veneziano, D.,(1982). Seismic Safety of Reinforced Concrete Members and Structures. *Earthquake Engineering Structural Dynamics*, Vol. 10, 1982,pp 179-193.

Park, Y. J., and Ang, A.-S. (1985). Seismic Damage Analysis of Reinforced Concrete Buildings. *Journal of Structural Engineering, ASCE*, Vol. III, No. 4, pp740-757.

DiPasquale, E., and Cakmak, A. S. (1990). Seismic Damage Assessment using Linear Models. *Soil Dynamic and Earthquake Engineering*, 9(4), pp 194-215.

Kircher, C.A., Nassar, A.A., Kustu, O.,Holmes, W.T., (1997), Development of Building Damage Functions for Earthquake Loss Estimation. Earthquake Spectra, EERI, November 1997, Vol. 13, N.4, pp663-683.

O'Rourke, M. J., and So, P. (2000). Seismic Fragility Curves For On-Grade Steel Tanks. *Earthquake Spectra*, Volume 16, No.4 (November 2000), pp801-815.

Singhal, A., and Kiremidjan, A. S. (1996). Method for Probabilistic Evaluation of Seismic Structural Damage. *Journal of Structural Engineering, ASCE*, Vol 122, No.12(December, 1996), pp 1459-1467.

Ichii, K. (2004), Fragility Curves for Gravity-Type Quay Walls based on Effective Stress Analyses. *13th World Conference on Earthquake Engineering*, Vancouver, B.C., Canada, August 1-6, 2004, paper No.3040.

Holden,T., (2001), Seismic Performance of Precast Reinforced and Prestressed Concrete Walls. *Journal of Structural Engineering*, ASCE (2001) ,pp286-296.

Liyanage, U. L. (2004). Biaxial Lateral Loading Behaviour of Thin Concrete Walls, Master Thesis, University of Canterbury, Christchurch, New Zealand.

FEMA 273:(1997), NERHP Guidelines for the Seismic Rehabilition of Buildings and FEMA 274:(1997) Commentary. Federal Emergency Management Agency, Washington, U.S.A.

Mander, J. B.(2003) Beyond Ductility: The Quest Goes On. *Symposium to Celebrate the Lifetime Contributions of Professors Emeriti Tom Paulay and Bob Park*, Christchurch, New Zealand, pp75-86.

Kennedy, R.P.,Cornell, C.A., Campbell, R.D., Kaplan, S. and Perla, H.F.(1980). Probabilistic Seismic Safety Study of an Existing Nuclear Power Plant.' *Nuclear Engineering and Design*, No.59, pp5-338.

Dutta, A., and Mander, J.B. (1998), Capacity Design and Fatigue Analysis of Confined Concrete Columns. Technical Report. MCEER-98-0007, Multidisciplinary Centre for Earthquake Engineering, State University of New York, Buffalo, NY, U.S.A

Pekcan G., Mander, J.B, Chen, S., (1999), Fundamental Considerations for the Design of Non-Linear Viscous Dampers. *Earthquake Engineering and Structural Dynamics*, N28. pp1405-1425.

HAZUS 99-SR2. (2004). Earthquake Loss Estimation Methodology. HAZUS Technical Report, Federal Emergency Agency and National Institute of Buildings Science, Washington D.C., http://www.fema.gov/hazus/(accessed 4/4/10).

NZS 3101: 1995: Concrete Structures Standard, Part 1: The Design of Concrete Structures, Standards Association of New Zealand Wellington, 1995.

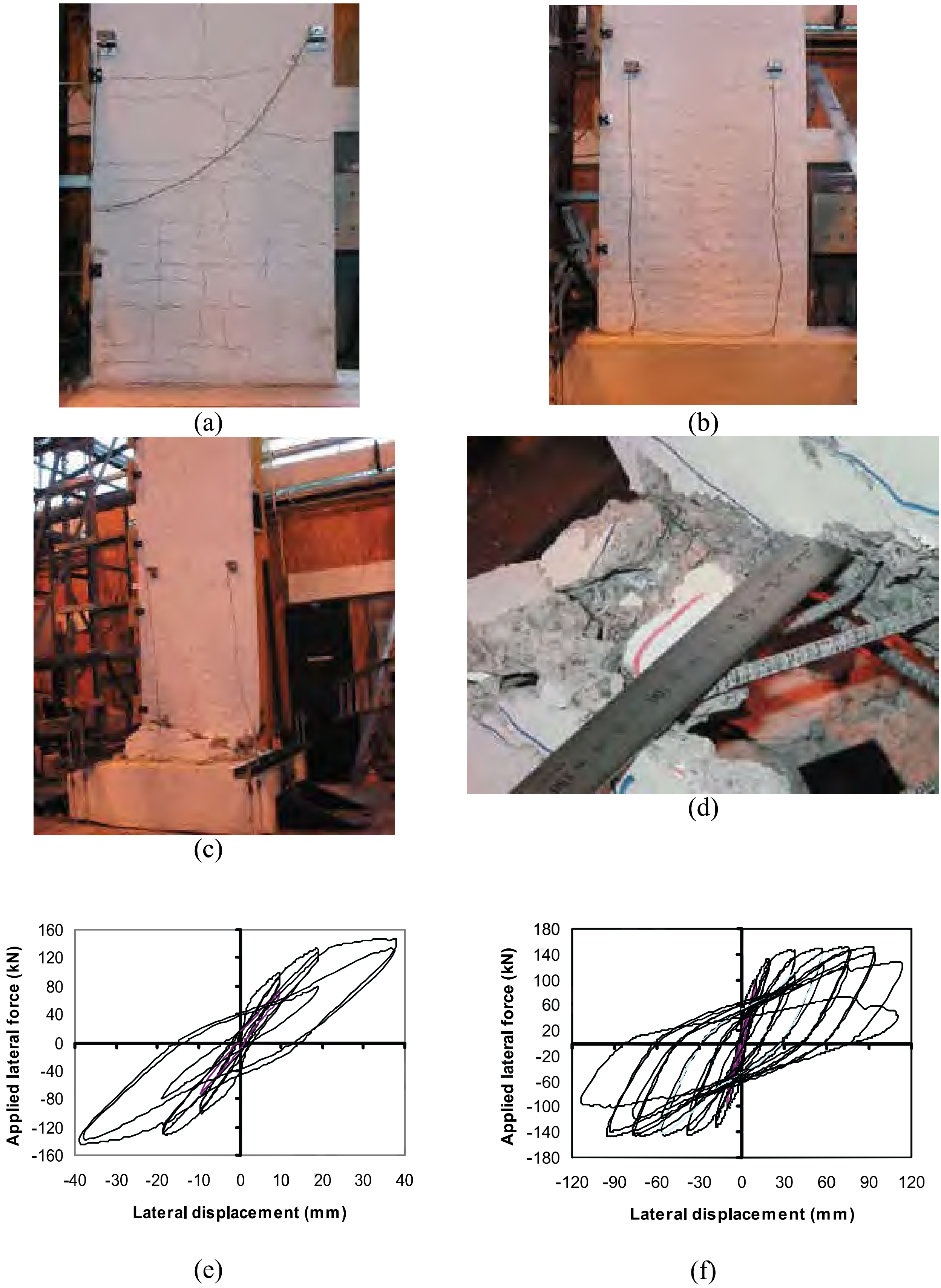


Figure 1. The seismic performance of conventional reinforced concrete walls according to the standard code of NZ3101 tested under quasi-static in-plane loading: (a) cracks on the surface of the wall at 0.5% drift; (b) more and wider cracks occurred at 1.0% drift; (c) spalling and crushing on both bottom corners of the wall at 2.0% drift; (d) fractured longitudinal bars at 3.0% drift; (e) the hysteresis loop at 1.0% drift; and (f) the hysteresis loop at 3.0% drift.

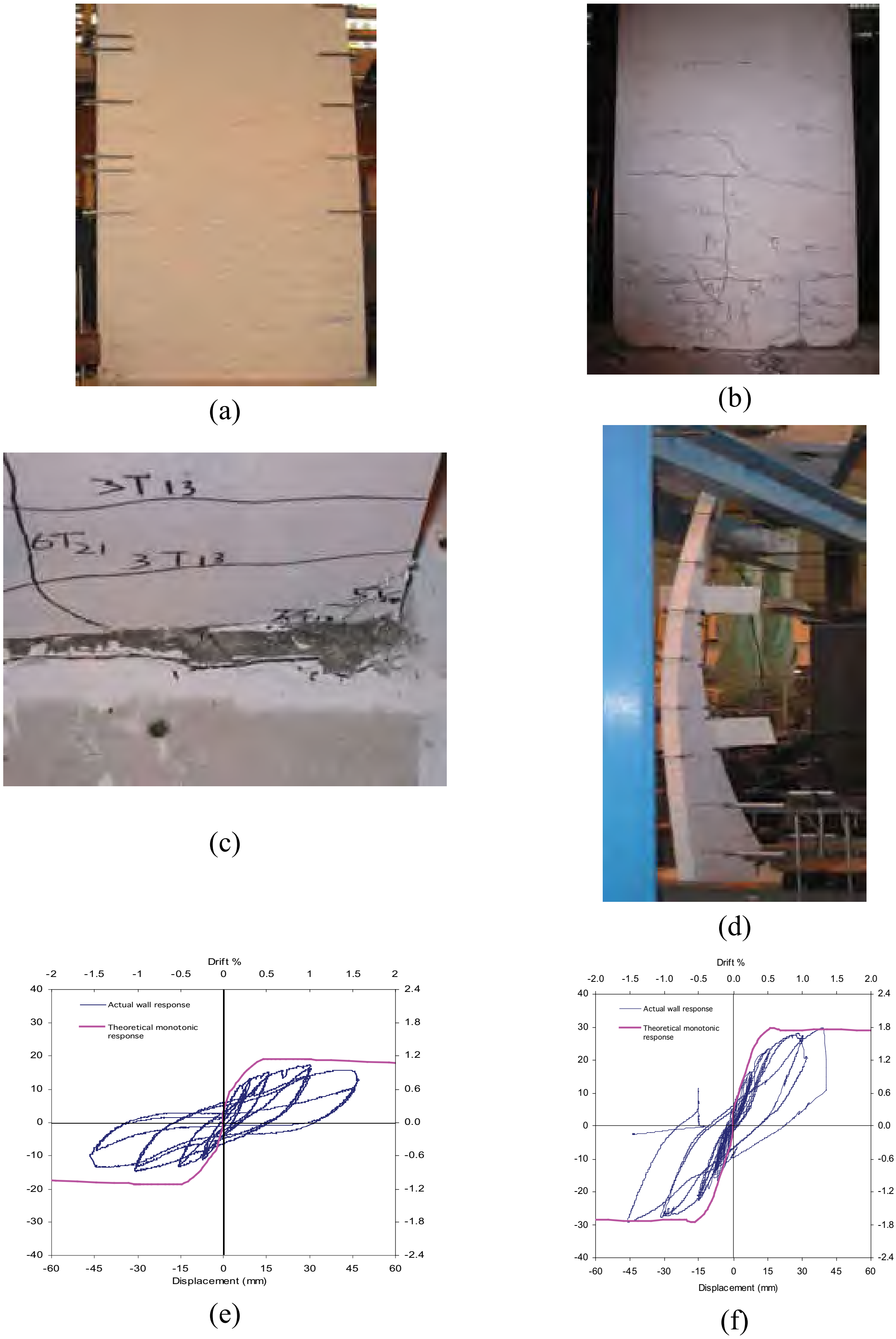


Figure 2. The seismic performance of slender precast wall panels under biaxial loading: (a) cracking of concrete at the bottom of the wall at 1.0% drift; (b) the spalling and crushing at the eastern corner of wall at 1.5% drift; (c) the fracturing of the outermost longitudinal bars at 2.0% drift and shifted the wall 30mm from the original place; and (d) lateral-torsional buckling failure at 2.0%; (e) in-plane hysteresis loops at 1.5% drift; and (f) out-of-plane hysteresis loop at 1.5% drift.

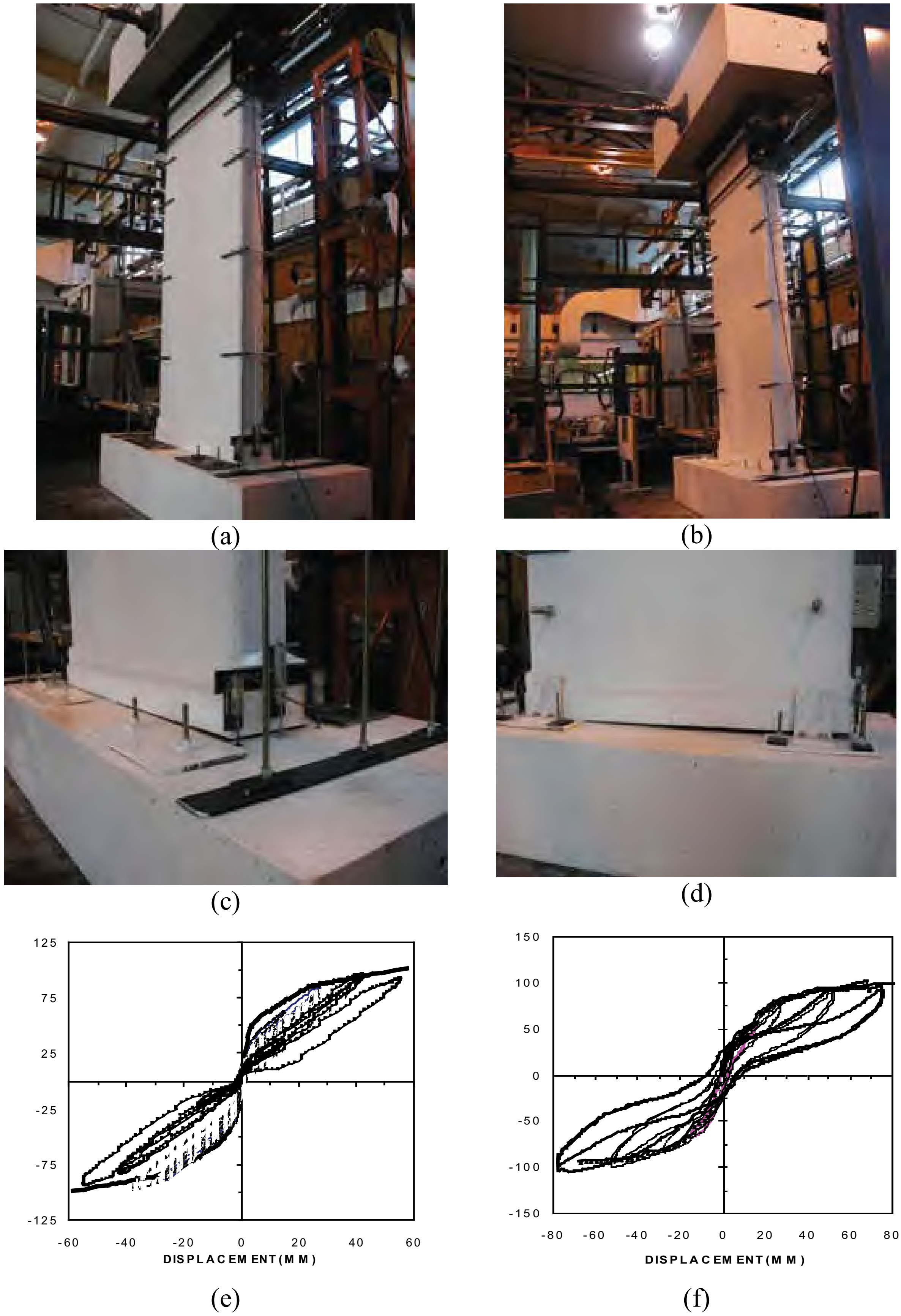


Figure 3. The seismic performance of precast hollow core walls designed according to damage avoidance design philosophy subjected to biaxial loading; (a) the biaxial behaviour of wall at 1.0% drift; (b) overall behaviour of wall at 2.0% drift; (c) no visible damage at 1.5% drift; and (d) the uplift of the eastern corner of wall did not cause any cracking, crushing and spalling of concrete; (e) the hysteresis loop at 2.0% drift; and (f) the hysteresis loop at 3.0% drift.

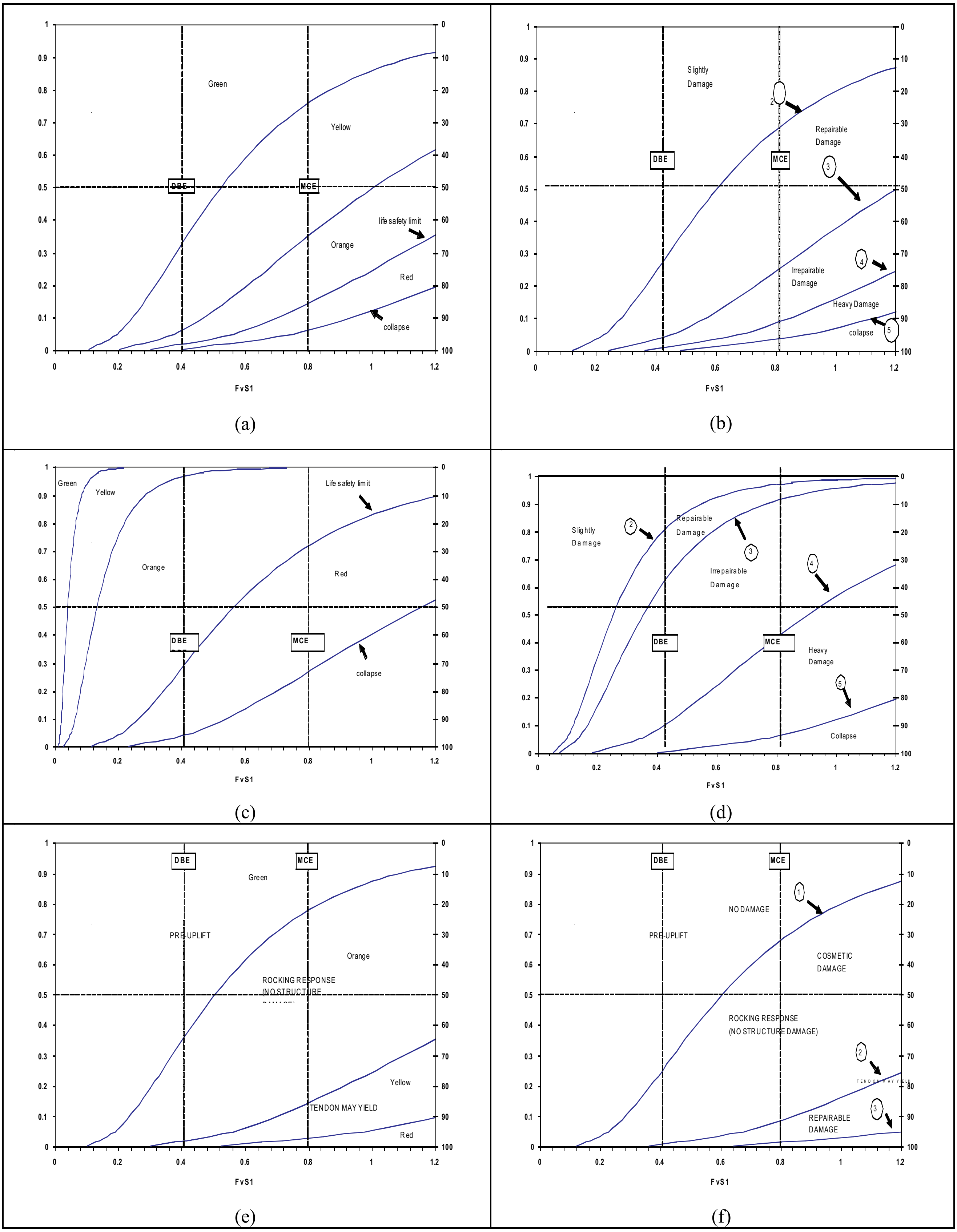


Figure 4. Fragility curves for three different types of wall using colour-coded and damage states number format: (a) the conventional precast wall followed NZ3101 standard; (b) slender precast wall panels; (c) precast hollow core wall using damage avoidance design philosophy; (d) colour-coded damage for slender precast wall; (e) precast hollow core walls; and (f) colour-coded damage for precast hollow core.

PERFORMANCE OF HIGH STRENGTH FLOWING CONCRETE CONTAINING STEEL FIBRE

Mahyuddin Ramli[1], Eethar Thanon Dawood[2]
[1] *Professor in Building Technology, School of Housing, Building and Planning, Universiti Sains Malaysia, 11800 Penang- Malaysia*
[2] *PhD Student in Building Technology, School of Housing, Building and Planning, Universiti Sains Malaysia, 11800 Penang- Malaysia*

Abstract
This study investigates the use of varying percentages of steel fibre (0-2%) as volumetric fractions on the high strength flowing concrete (HSFC) to determine the density, compressive strength, flexural strength, splitting tensile and static modulus of elasticity. The flexural toughness indices were also determined according to ASTM C1018. The results show that the use of 1.0% steel fibre increases the compressive strength by about 10%, whereas, the use of 1.5% of steel fibre increases the flexural strength by about 42%. The splitting tensile strength, static modulus of elasticity & toughness indices results indicate that the increase in steel fibre fractions leads to improvements in these properties as well.

Keywords: *Fibre concrete, Performance, Steel fibre, high strength flowing concrete*

INTRODUCTION

The mechanical properties of concrete can be improved by randomly oriented short discrete fibres which prevent or control initiation, propagation, or coalescence of cracks. For concrete consisting of hardened cement paste and aggregates of different sizes, micro cracks are among the intrinsic factors which can be overcome by using fibre reinforcement. The main disadvantage of incorporating a fibre is the loss of workability and the increased difficulty of casting. This status mostly results in insufficient workability and high volumes of entrapped air in concrete which leads to reduction in its strength and durability (Felekoğlu et al., 2007). The modern concrete can be designed to have a high workability that allows the concrete to flow in the congested reinforcement areas and fill complicated formwork without segregation (Gang et al., 2008, Okamora et al 2003, Khayat et al., 2002).

The character and the performance of fibre reinforced concrete (FRC) are related to the properties of concrete and the fibres. The properties of fibres that are usually of interest are fibre concentration, fibre geometry, fibre orientation, and fibre distribution. Steel fibre has a considerably larger length and higher Young's modulus of elasticity as compared to the other fibre-types. This leads to an improved flexural rigidity and has great potential for crack control, although the volumetric density is high (Bentur & Mindess, 1990). Many researchers have conducted investigations to study fibre reinforced concrete in the past. Shah & Naaman (1976) determined the tensile strength, flexural strength and compressive strength tests on mortar specimens reinforced with steel fibre and found that the tensile and flexural strength of steel fibre reinforced mortar was at least twice to three times that of plain mortar specimens. Similarly, the effect of adding steel fibres to the concrete or mortar on the compressive strength ranges from negligible and sometimes up to 25%, whereas considerable increase in the flexural and toughness of the material has been observed. (Nataraja et al. 1999).

Therefore, mix proportions are proposed here to improve the rheological property of HSFC used as a building material. The incorporating of different volume fractions of steel fibre ranging from low volume fractions till the high volume fraction (0-2%) was applied here to study their effects on the properties of high strength flowing concrete (HSFC) in fresh and hardened concrete status.

MATERIALS AND MIX PROPORTIONS

Materials

The cement used in concrete mixtures was ordinary portland cement type I from Tasek Corporation Berhad. Silica fume was obtained from Scancem Materials Sdn. Bhd. and was used as partial replacement of cement. The chemical compositions of ordinary portland cement and silica fume are stated in Table 1.

The superplasticizer (SP) is Conplast SP1000 obtained from Fosroc Sdn. Bhd. and was used to establish the desired workability of mixes. The fine aggregate was natural sand, with fineness modulus of 2.86 and maximum size of less than 5 mm. The steel fibres with hooked ends were supplied by Hunan Sunshine Steel Fibre Co. Ltd, and their mechanical properties are presented in Table 2.

Table 1. Chemical composition of ordinary Portland cement and silica fume

Constituent	Ordinary Portland Cement	Silica fume
	% by weight	% by weight
Lime (C_aO)	64.64	1.0% (max)
Silica (S_iO_2)	21.28	90% (max)
Alumina (Al_2O_3)	5.60	1.2 % (max)
Iron Oxide (Fe_2O_3)	3.36	2.0% (max)
Magnesia (M_gO)	2.06	0.6%(max)
Sulphur Trioxide (SO_3)	2.14	0.5%(max)
N_2O	0.05	0.8%(max)
Loss on Ignition	0.64	6% (max)
Lime saturation factor	0.92	-----
C_3S	52.82	-----
C_2S	21.45	-----
C_3A	9.16	-----
C_4AF	10.2	-----

Table 2. Physical properties of Steel fibre

Fibre Properties	Quantity
Average fibre length,(mm)	30
Average fibre diameter,(mm)	0.56
Aspect ratio (L/d)	54
Tensile strength (MPa)	> 1100
Ultimate elongation (%)	< 2
Specific gravity	7.85

Mix Proportions

The concrete mix proportions are shown in Table 3. Fourteen concrete mixes were prepared using water-binder (Cement + Silica fume) ratio of 0.43 and silica fume replacement was 10%. The amount of cement, silica fume, sand and free water were kept constant. The amount of superplasticizer was varied from 1.8% to 2.2% by weight of binder materials to maintain the workability and the uniformity of the mixes. The mix design of the control mix (C0) is carried out according to the absolute volume method given by the American Concrete Institute (ACI 318 R) for the flowable high strength concrete. The steel fibres are added to the mixes according to the volumetric fractions of 0.25, 0.5, 0.75, 1.0, 1.25, 1.5, 1.75 & 2.0 % by weight of cement (mixes C1-C8).

Table 3. Concrete Mixes Design

Index	Cement Kg/m^3	Silica fume Kg/m^3	Sp%	Sand Kg/m^3	Gravel Kg/m^3	W+Sp/B	Steel fibre (SF) %	Slump flow (mm)
C0	500	50	1.8	880	715	0.43	0	650
C1	500	50	1.8	880	715	0.43	0.25	620
C2	500	50	1.8	880	715	0.43	0.50	600
C3	500	50	1.8	880	715	0.43	0.75	590
C4	500	50	2.0	880	715	0.43	1.0	610
C5	500	50	2.0	880	715	0.43	1.25	580
C6	500	50	2.0	880	715	0.43	1.50	560
C7	500	50	2.2	880	715	0.43	1.75	580
C8	500	50	2.2	880	715	0.43	2.0	550

EXPERIMENTAL PROGRAMME

Each test results are represented by three cube samples 100 mm for each test age to determine their density and compressive strength at various ages after undergoing water curing. The slump flow test for the mixes was performed according to JSCE-F503 (1990) with a targeted slump flow of 600 mm±50mm. The cube specimens were left in the moulds for 24 hours after casting at room temperature of 20 °C. After demoulding, the specimens were transferred into water for further curing until the age of the test. All specimens were tested at saturated surface dry condition and carried out according to BS 1881: Part 114. The compressive strength of each sample age was tested immediately after determining the density of the test specimens according to BS 1881: Part 116 for each test age. The splitting tensile strength test was done by using 100 × 200 mm concrete cylinders according to ASTM C496. The static modulus of elasticity test was achieved using 150 × 300 mm concrete cylinders according to ASTM C469. The toughness indices of the specimens were conducted using 100 × 100 × 500 mm samples conforming to ASTM C1018.

RESULTS AND DISCUSSION

Saturated surface dry density

Table 4 shows the results of the saturated surface dry density for all mixes. The saturated surface dry density at 28 days for the different mixes of steel fibre (C1-C8) show that there is a rise in the density level as the volume fraction of steel fibre increases and this is resulted from the specific gravity of steel fibre which increases the overall density of mortar as shown in Figure 1.

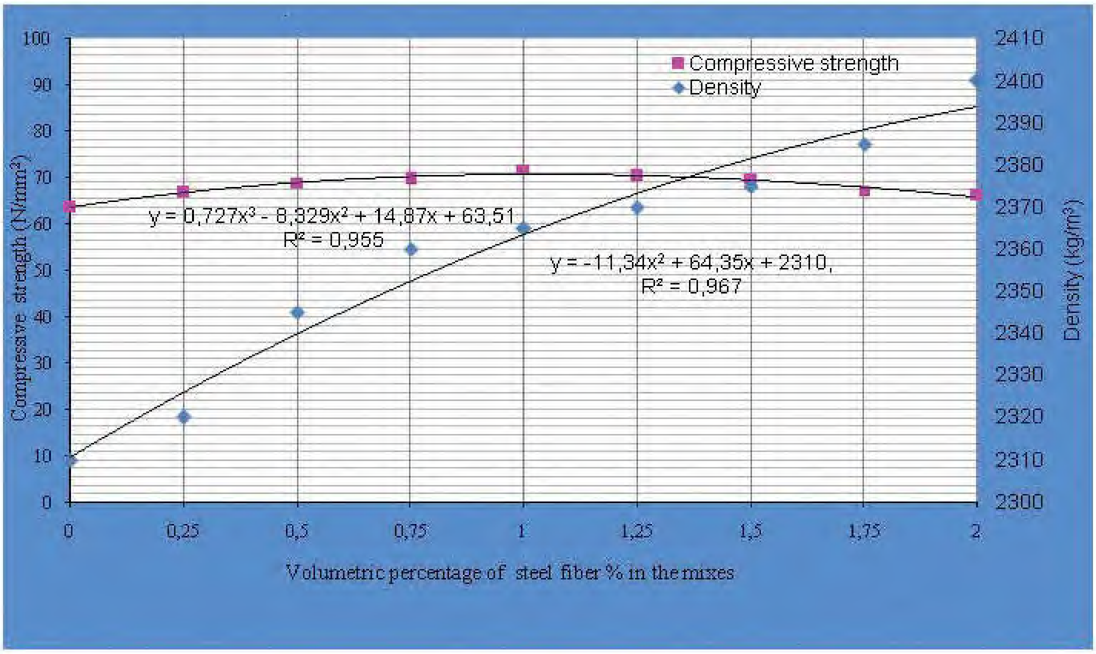


Figure 1. Relationship between volume fraction of steel fibre & density-compressive strength at 28 days for HSFC.

Compressive strength

From Table 4, the increase in the compressive strength of the high strength flowing concrete (HSFC) with steel fibre is up to 10 % compared with that of the control concrete. This result was obtained from the volume fraction of steel fibre up to 1.0 % used in the mix (C4). This condition can be attributed to the improvement in the mechanical bond strength when the fibres allow the ability to delay the micro-crack formation and arrest their propagation afterwards up to a certain extent (Steffen & Joost 2001). The use of more than 1.0% of steel fibre reduces the compressive strength and this can be attributed to the air voids and disintegration when the excessive fibre content was used in the mixes as shown in Figure 1.

Flexural strength

The flexural strength of the high strength flowing concrete mixes is shown in Table 4. The result of concrete mix containing the steel fibre up to 1.5% for the mix (C6) increases the flexural strength by about 42% higher than the control mix (C0) and this could be due to better compactness and homogeneity of fibre distribution in HSFC. It can also be noticed that the use of more than 1.5 % of steel fibre slightly decreases the flexural strength and the reason could lie behind the physical difficulties in providing a homogenous distribution of fibres within the concrete (Chen & Liu, 2004). Figure 2 illustrates the relationship between steel fibre fractions in the concrete mixes with the flexural strength.

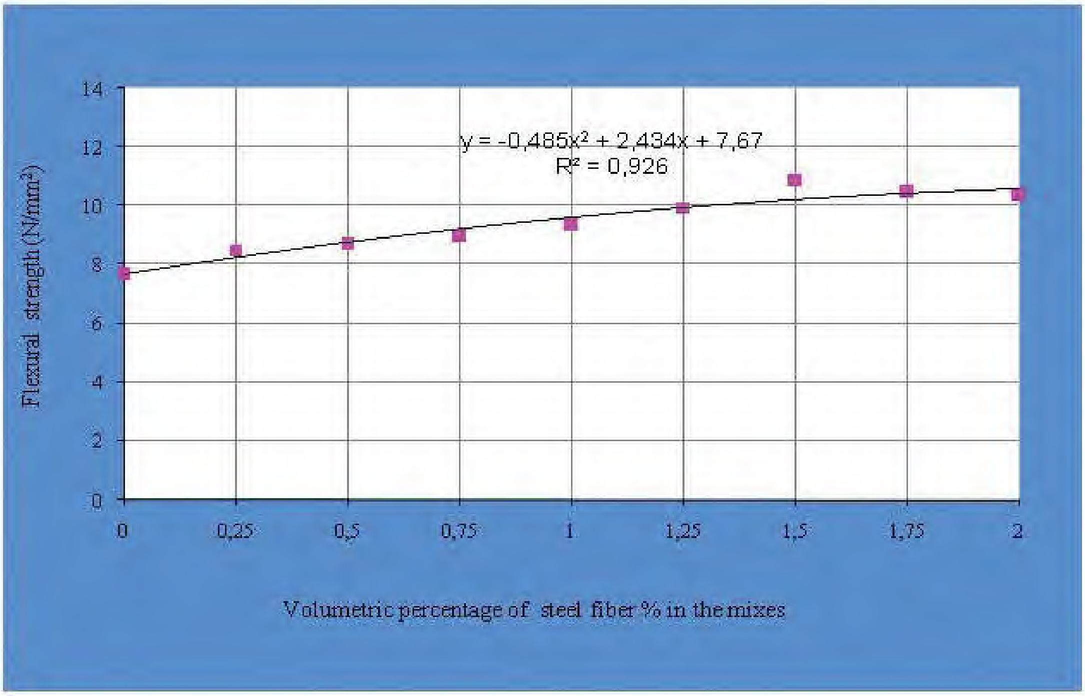


Figure 2. Relationship between volume fraction of steel fibre & flexural strength at 28 days for HSFC

Splitting Tensile

The splitting tensile strength of the flowable concrete mixes is shown in Table 4. The result of concrete mix containing the steel fibre up to 2% for the mix (C8) increases the splitting tensile strength of 58% higher than the control mix (C0) and this could be due to improved in toughness matrix, compactness and homogeneity of fibre distribution in HSFC (Chin & Liu, 2004). Figure 3 illustrates the relationship between steel fibre fractions in the concrete mixes with the splitting tensile strength.

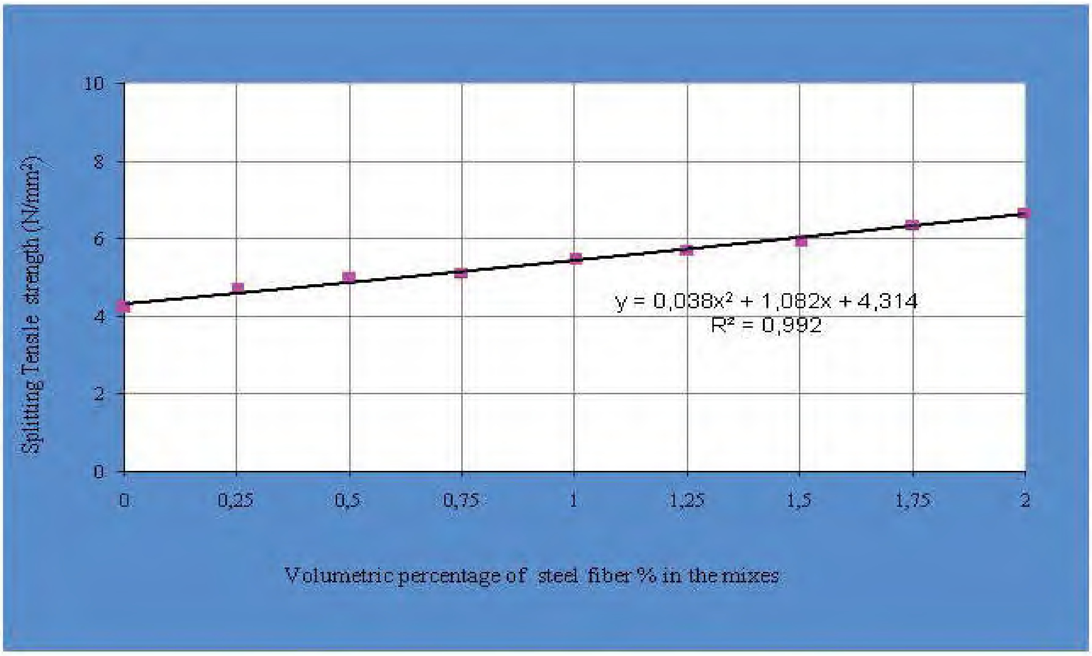


Figure 3. Relationship between volume fraction of steel fibre & splitting tensile strength at 28 days for HSFC

Static Modulus of Elasticity

The results of static modulus of elasticity are presented in Table 4. It can be concluded that the static modulus of elasticity increases with volume fraction of steel fibre as it's shown in Figure 4. The static modulus of elasticity is increased by about 32% by using 2% of steel fibre as volumetric fraction. The performance of the mechanical properties which utilize the different percentages of fibres is summarized in Figure 5.

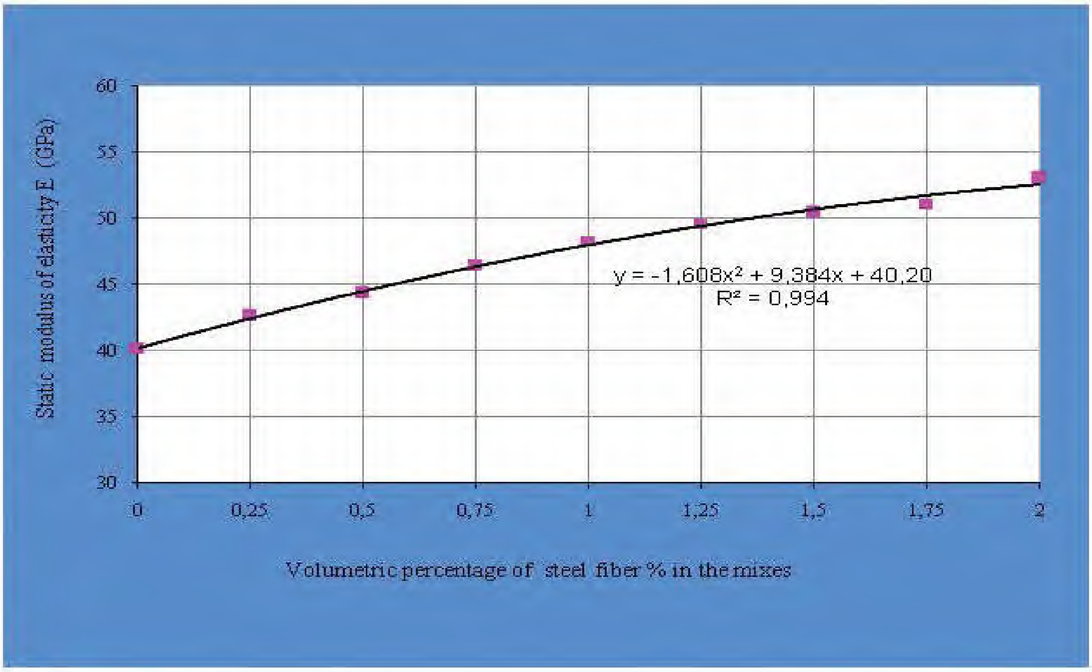


Figure 4. Relationship between volume fraction of steel fibre & static modulus of elasticity at 28 days for HSFC

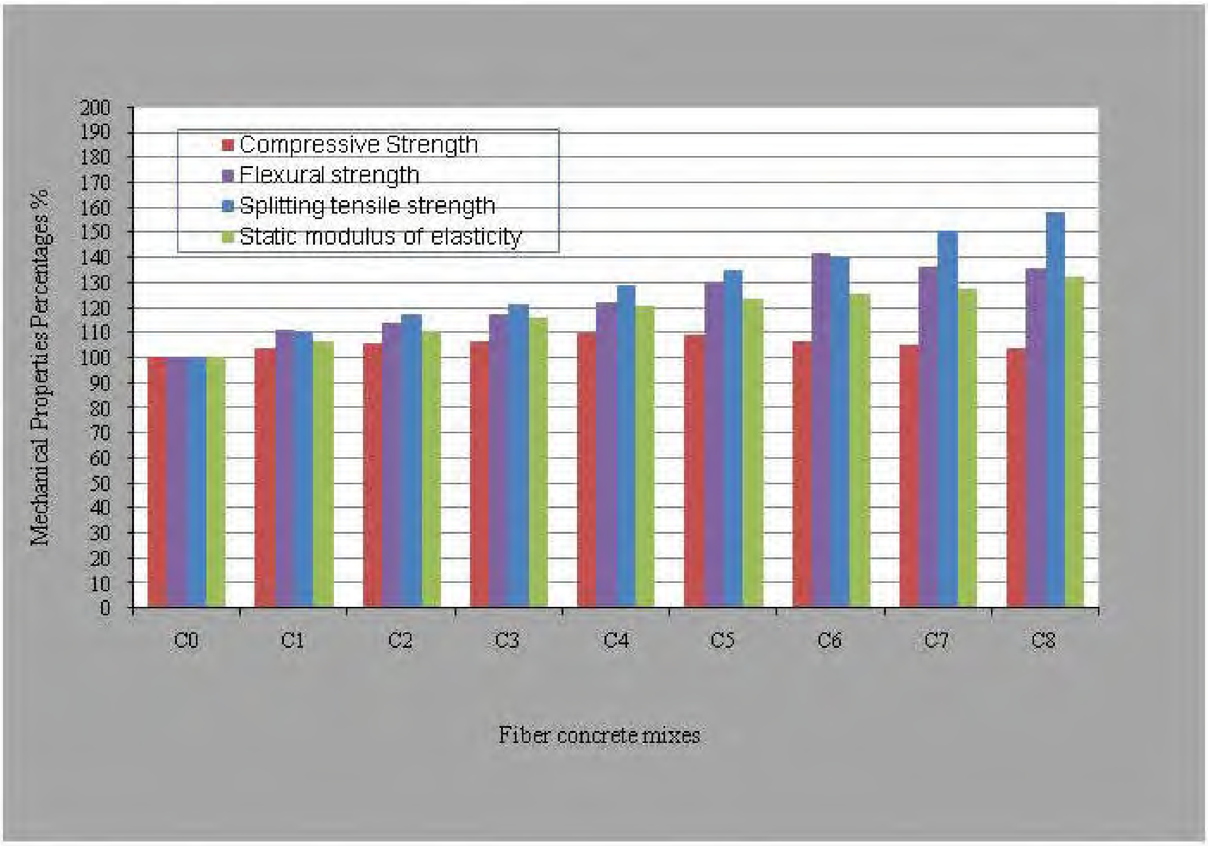


Figure 5. Relationship between fibre concrete mixes with mechanical properties percentage at 28 days.

Toughness Indices

Toughness is also defined as the energy absorption capacity. The toughness indices can be determined according to ASTM C1018. The indices I-5 and I-10 can be calculated from this test where the ratio of the area under the load deflection curves up to 3 and 5.5 times the first crack deflection, divided by the area up to the first crack deflection respectively as shown in Figure 6. Table 4 illustrates the results of the I-5 & I-10 for 28 days. It can be observed that the steel fibre has a clear effect when the volume fraction used is 1.0 % and more. The toughness indices are found to increase with the increasing fibre content, and the behaviour indicates the ability of fibres in arresting cracks at both micro- and macro–levels. At micro-level, fibres inhibit the initiation of cracks, while at macro-cracks; fibres provide effective bridging and impart sources of toughness and ductility (Balaguru & Shah, 1992, Banthia & Sappakittipakorn, 2007). This was also supported by other researchers (Nataraja et al., 1999), where the flexural toughness can be increased as the fibre volume fraction is increased and similarly higher values of the toughness indices can be achieved at higher fibre volume fractions. The Figure 7 shows the effects of using different volume fractions of steel fibre on the toughness indices at 28 days of normal water curing.

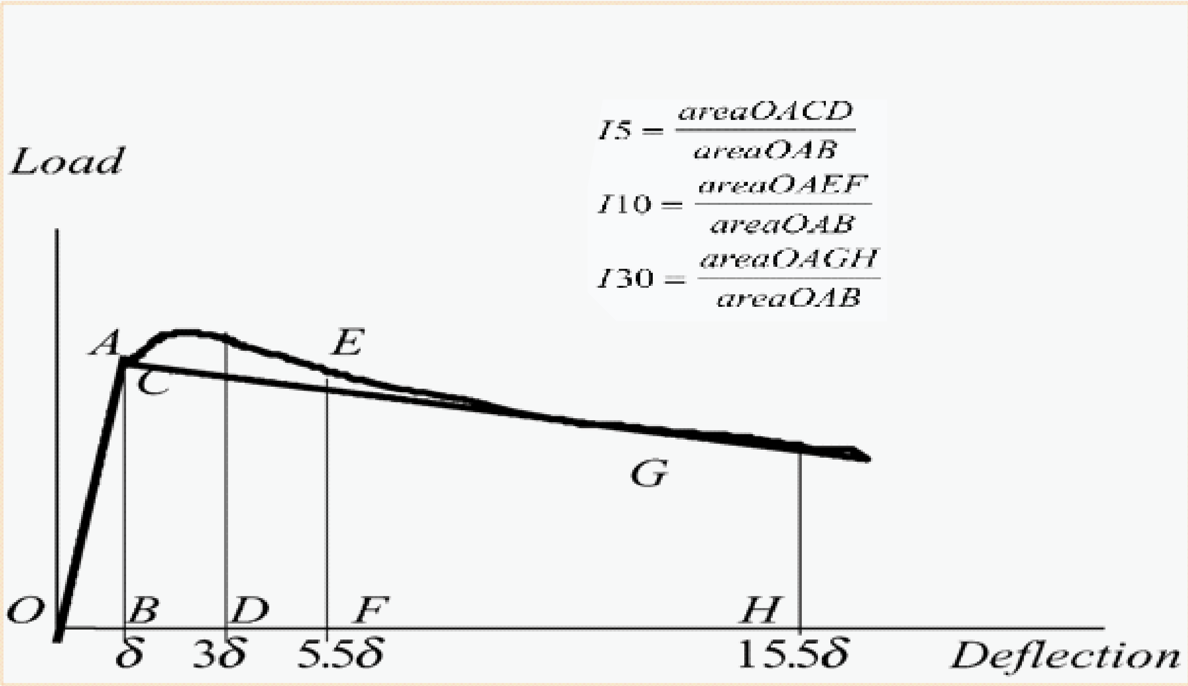


Figure 6. Toughness indices according to the testing method ASTM C1018

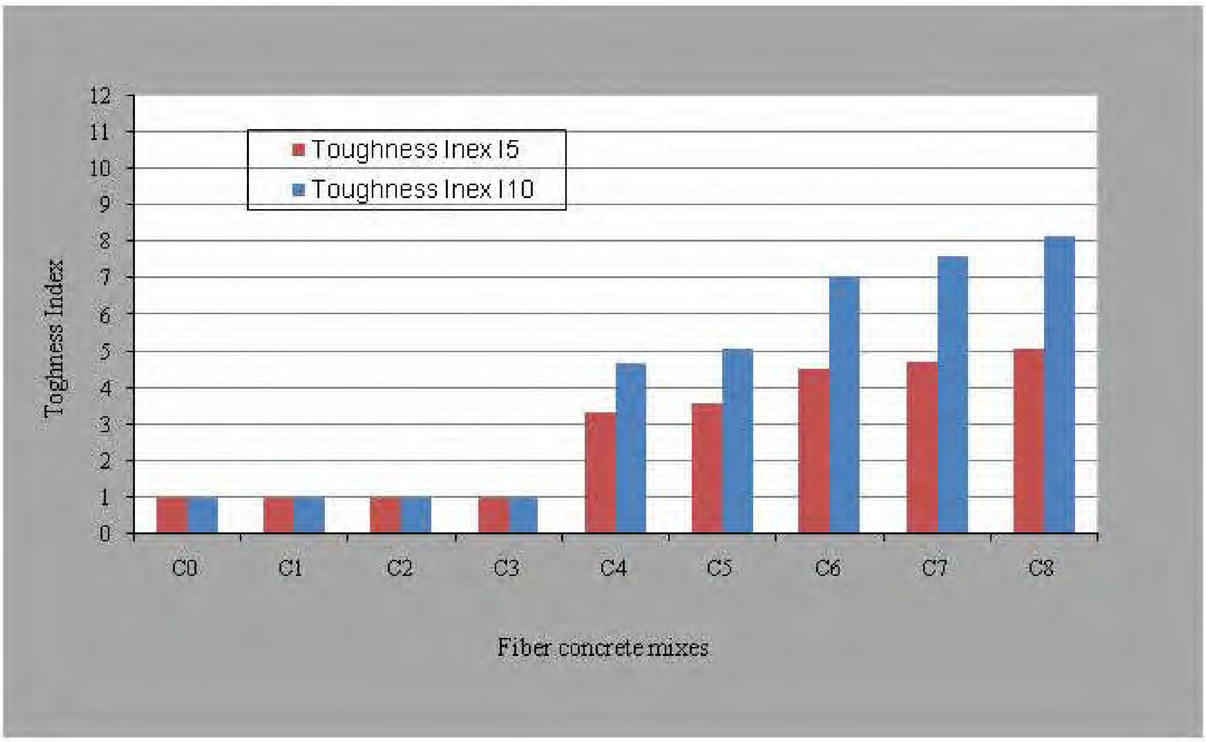


Figure 7. Relationship between fibre concrete mixes with toughness index at 28 days.

Table 4. Mechanical Properties of Concrete Specimens

Index	Density Kg/m^3 (28 days)	Compressive strength (MPa) (28 days)	Flexural strength (MPa) (28 days)	Splitting Tensile Strength (MPa) (28 days)	Static modulus of elasticity (GPa) (28 days)	Toughness Index (I -5) (28 days)	Toughness Index (I -10) (28 days)
C0	2290	63.6	7.65	4.22	40.1	1	1
C1	2320	66.7	8.50	4.67	42.6	1	1
C2	2345	68.9	8.70	4.95	44.3	1	1
C3	2360	69.6	8.95	5.12	46.4	1	1
C4	2365	71.8	9.35	5.45	48.3	3.32	4.64
C5	2370	70.3	9.95	5.70	49.6	3.58	5.05
C6	2375	69.3	10.85	5.92	50.4	4.53	7.05
C7	2385	67.2	10.45	6.36	51.1	4.68	7.58
C8	2400	66.1	10.35	6.67	53.0	5.05	8.12

CONCLUSIONS

The results of an experimental study on the high strength flowable concrete reinforced with various volume fractions of steel fibre reveal the following conclusions:

The use of steel fibre increases the density of HSFC due to specific gravity of the fibre.

The compressive strength results show that the use of 1.0% steel fibre increases the compressive strength by about 10%, and this is maybe due to the improvement in the mechanical bond strength when the fibres allow the ability to delay the micro-crack formation and arrest their propagation afterwards up to a certain extent.

The flexural strength of concrete mixes containing steel fibres increases with the increasing volume fraction. The highest values for these properties obtained as the 1.5% of steel fibre included in the mix.

The Splitting tensile strength and static modulus of elasticity (Ec) of concrete mixes containing steel fibres increases with the increasing volume fraction. The use of 2% of steel fibre increases the splitting tensile and the static modulus of elasticity by about 58% and 32% respectively.

The toughness indices results illustrate that the use of less than 1% of volume fraction of steel fibre has no effect in improving the toughness indices I 5 & I 10, whereas the use of 1% and more leads to improve the ductility and absorption capacity.

ACKNOWLEDGEMENT

The work described in this paper was a part of PhD research program of the 2nd author which is supported by a research grant and Fellowship from the Universiti Sains Malaysia.

REFERENCES

ACI American Concrete Institute (ACI 211.1-91) Standard P (2000) Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete, ACI Manual of Concrete Practice, Part 1: Materials and General Properties of Concrete.

American Society Tests of Material, ASTM C469, C496, C642, & C1018 Annual Book of ASTM Standard, Volume 04.02 ASTM international, West Conshohocken, PA, www.astm.org.

Balaguru P. N. & Shah S. P. (1992) Fibre Reinforced Cement Composites, McGraw-Hill Inc., New York.

Banthia N. & Sappakittipakorn M. (2007) Toughness Enhancement in Steel Fibre Reinforced Concrete Through Fibre Hybridization, Cement & Concrete Research; 37, pp. 1366-1372.

Bentur A. & Mindess S. (1990) Fibre Reinforced Cementitious Composites, *Elsevier Applied Science*, London, UK.

Chen B & Liu J (2004) Residual Strength of Hybrid-Fibre-Reinforced High-Strength Concrete After Exposure to High Temperatures Cement & Concrete Research;34. pp.1065-69.

Felekoğlu B., Turkel S. & Altuntas Y. (2007) Hybrid Fibre Feinforced Self-Compacting Concrete with High-Volume Coarse Fly Ash, Const. & Build. Mater. ; 21, pp.150-156.

Felekoğlu B., Turkel S. & Altuntas Y. (2007) Effects of Steel Fibre Reinforcement on Surface Wear Resistance on Self Compacting Repair Mortar, *Cement & Concrete Compsite*; 29, pp.391-396.

Gang L., Wang K & Rudolphi T. J. (2008) Modelling Rheological Behavior of Highly Flowable Mortar Using Concepts of Particle and Fluid Mechanics, Cement & Concrete Composite. ;30, pp.1-12.

Hannant D. J. (1987) Fibre Cement and Fibre Concrete Chichester, UK:Wiley.

JSCE-F503, (1990) Method of Test for the Slump Flow of Concrete, Standards of Japan Society of Civil Engineers, Tokyo.

Khayat K. H. ,R. Morin R (2002) Performance of Self-Consolidating Concrete Used to Repair Parapet Wall in Montreal, Proceedings of the first north American conference on the design and use of self-consolidating concrete, pp. 475-481.

Nataraja M. C., Dhang N. & Gupta A. P. (1999) Stress-Strain Curves for Steel Fibre Reinforced Concrete Under Compression, Cement & Concrete Composite; 21, pp. 383-390.

Neville A. M. Properties of Concrete, (1995), Fourth and Final edition.

Okamura H., & Ouchi M. (2003) Self Compacting Concrete, *J. Advance Concrete Technology*, 1(1); pp.1-15.

Shah S. P. & Naaman A. E., 1976 Mechanical Properties of Glass and Steel Fibre Reinforced Mortar *ACI Journal* ;73(1): pp.50-53.

Steffen G. & Joost C. W(2001) Parameter Study on the Influence of Steel Fibres and Coarse Aggregate Content on the Fresh Properties of Self Compacting Concrete *Cement & Concrete Research*; 31, pp.1793-1798.

PERFORMANCE OF GROUTED SLEEVE CONNECTORS SUBJECTED TO INCREMENTAL TENSILE LOADS

Ahmad Baharuddin Abd. Rahman[1], Ling Jen Hua[2], Izni Syahrizal Ibrahim[2], Zuhairi Abd. Hamid[3]
[1]Corresponding author, Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor, Malaysia.
[2]Faculty of Civil Engineering, Universiti Teknologi Malaysia, 81310 Skudai, Johor, Malaysia
[3]Construction Research Institute of Malaysia (CREAM), Kuala Lumpur, Malaysia

Abstract
Full strength grouted sleeve connectors ensure the integrity of connected precast concrete components. This research investigated the behaviour and the strength performance of the proposed A, B, C and D-series grouted sleeve connectors for joining precast concrete components. The connectors were subjected to increasing tensile loads until failure. The performances of the connectors were also evaluated in terms of stiffness, yield strength, ductility and failure modes. The experimental results show that the C-series grouted splice sleeve connectors successfully achieved the full tensile strength of the connected steel bars. In addition, the confinement provided by the steel sleeve controls and delays the splitting cracks of the surrounding grout and eventually enhances the bond between bar-and-grout significantly. The enhanced bond contributes to a shorter development length of the connected bar as compared to the conventional bar lapping method.

Keywords: *bond, connector, grout, precast concrete, splice sleeve*

INTRODUCTION

A sleeve combined with grout is a cylindrical connector used to join two discontinuous steel reinforcement bars without complying to the conventional lap length. The use of conventional bar lapping system in precast concrete structures may be uneconomical and unpractical, particularly for large diameter steel bars because they require longer development length and lead to congestion of steel bars, and also difficulties in manufacturing fabrication, and transportation. Alternatively, splice sleeves can be used as a connection system for connecting reinforcement bars extruding from one structural element to other elements. Apart from wall frame systems, splice sleeves can also be used as beam-to-column, column-to-column, and column-to-base connections (Figure 1) This method is widely used in North America since the late 1980's (Jansson et al., 2008).

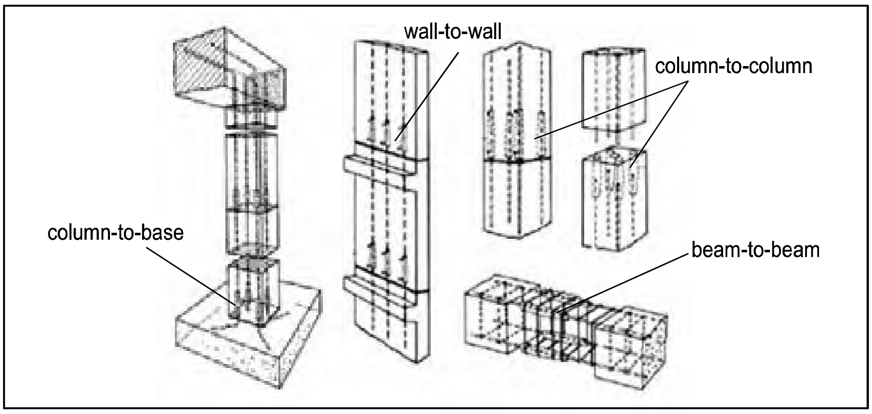


Figure 1. Application of grout-filled splice (source Splice Sleeve North America, Inc.)

A splice sleeve connector is a specially designed cylindrical sleeve that utilizes non-shrinkage grout as the bonding material to splice steel bars. The usage of splice sleeves is fairly simple and straightforward, where they are cast together with the precast concrete components, splicing the reinforcement bars protruding from other elements as they are jointed together at site. Then, the grout is pumped or poured into the sleeve, through the splice inlet, to fill the space and to ensure composite action among the components in the splice sleeves. As the grout reaches sufficient compressive strength of about 20N/mm^2, the temporary bracing that keep the precast elements in position can be removed.

The splice sleeves that are currently available in the market include *Lenton Interlok®*, *NMB Splice Sleeve®*, *Zap Screwlok©*, and Lenton Quick Wedge©. The connectors are proprietary products owned by foreign companies, namely *ERICO, Inc., Splice Sleeve North America, Inc.* and *BarSplice Product, Inc.,* accordingly. Their designs are complex and require special techniques to mould them into their intended shapes, of which this technique is rather expensive in Malaysia.

Figure 2. Available proprietary splice connectors

The objectives of the study were (a) to develop new splice sleeve connectors by utilizing commercially available steel section as a mean to reduce manufacturing cost and (b) to study their tensile performance and to determine the feasibility of the proposed splice sleeves. Fifteen specimens were tested and their performances were evaluated based on their stiffness, yielding strengths, ultimate tensile capacities, ductility and the failure mode.

LITERATURE REVIEW

In order to develop a splice sleeve with the least required development length, general concepts of bond and confinement are studied. It is known that from the literature review, bond performance between a steel bar and concrete can be enhanced through the confining concrete. This behaviour has been studied by several researchers, either experimentally or analytically.

Lutz (1966), Goto (1971) and Thompson (2002) discuss the principle of bond performance. The bond between steel bar and concrete is contributed by three major factors, (a) chemical adhesion, (b) friction, and (c) mechanical interlocking between bar ribs and concrete keys.

Regarding the effects of confinement, Untrauer (1965) found that the bond strength between steel and concrete increases linearly with normal pressure (confinement). He derived an equation that represents the relationship between the compressive strength of concrete, normal pressure and reinforcing bond strength. Soroushian (1989), who investigated the local bond stress behaviour of deformed bars in confined concrete, concluded that the bond strength decreased linearly as the bar diameter increases. Nilson (1975) derived a relationship between bond stress and bar slip in reinforced concrete to study the slip of bar in unconfined concrete. He developed a method to determine the slip from the displacement function by numerical integration of the strain.

Coogler (2006) conducted experimental testing for his master thesis, evaluating the tensile performance of two types of offset splice, namely *Lenton Quick Wedge*©, and *Zap ScrewLok*©. He employed these splices in full scale simply supported beams to study the loading behaviour.

Jansson (2008) evaluated the performance of two types of in-line splice, *Lenton Interlok*® and *NMB Splice Sleeve*® under incremental static and 1 million cycle fatigue loads. The tests were monitored under the *Michigan Department of Transportation* to determine the suitability of the splices.

Amin Einea (1995) developed new splices by utilizing commercially available pipe sections for field connections. A total of 15 grout-filled pipe splices were tested. He claimed that adequate bond strength could be achieved with the embedded length of seven times the bar diameter, provided that appropriate grout compressive strength and confinement are provided.

DESCRIPTION OF SAMPLES AND TESTING

In this paper, the discussions emphasize on the tensile test results of 15 specimens selected from several different series of splice sleeve connections. Figure 3 illustrates the details of the sleeve specimens. Only Y16 high strength steel bars were investigated, as this bar size is commonly used in the construction sites. High early strength non-shrinkage *Sika Grout-215* was used for the bonding material. The embedded length of the reinforcement bars in the sleeves was fixed at less than 150mm. Therefore, most of the overall length of the sleeve was limited to 300mm.

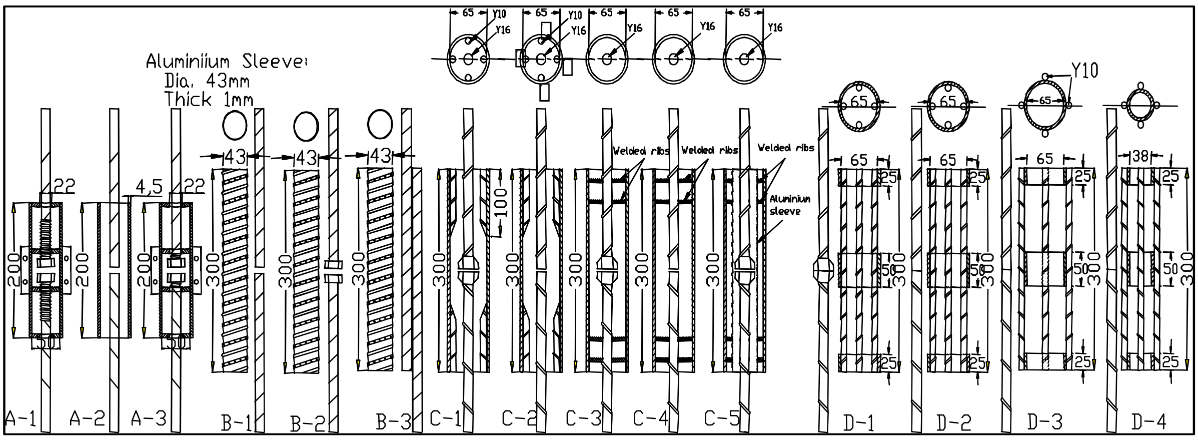


Figure 3. Detail of the test specimens

Table 1 shows the material properties and dimension of the test specimens. Detail descriptions of the specimens are as follows:

- ***A-Series:*** In specimens A-1 and A-3, each sleeve consists of two semi-circular steel pipes complete with two steel plates welded onto the pipes at 75mm from both ends, while each steel bars were threaded at the end to fix the nut. The nut-headed steel bars were joined head to head in the sleeve by grouting. As for A-2, no modification was made onto the pipe sleeves to receive the insertion of deformed steel bars.

- ***B-Series:*** The deformed steel bars and nut-headed steel bars were inserted into the 300mm corrugated aluminium sleeves from both ends to meet at mid length of B-1 and B-2, respectively. As for B-1, steel bars were lapped adjacently throughout the length of the sleeve.

- ***C-Series:*** Four Y10 steel bars were welded to the inner surface of the 65mm-diameter steel pipes from both ends, providing interlocking mechanism for the grout. As for C-3, C-4 and C-5, 2@2mm height rings were welded onto the pipes from both ends to interlock with the grout. Connector C-5 had additional aluminium sleeve placed between the steel bars and the mild steel pipe. The hexagonal tapered nuts were welded onto the Y16 high strength steel bars (Figure 4) before the nut-headed steel bars were being spliced in C-1, C-3 and C-5. Meanwhile, the deformed steel bars were spliced in C-2 and C-4 with 150mm to be embedded in the sleeves.

- ***D-Series:*** In these connectors, 4@300mm long Y10 steel bars were welded to the short pipes, of 50mm and 25mm at mid span and both end, respectively, to form a cylindrical sleeve. The short pipes of 65mm inner diameter were used for D-1, D-2 and D-3, while 38mm inner diameter short pipes were used for D-4. D-1 and D-2 had their Y10 steel bars welded onto the inner surface of the short pipes, while Y10 steel bars were welded on the outer surface of the short pipes.

Table 1. Material properties and dimension of the specimens

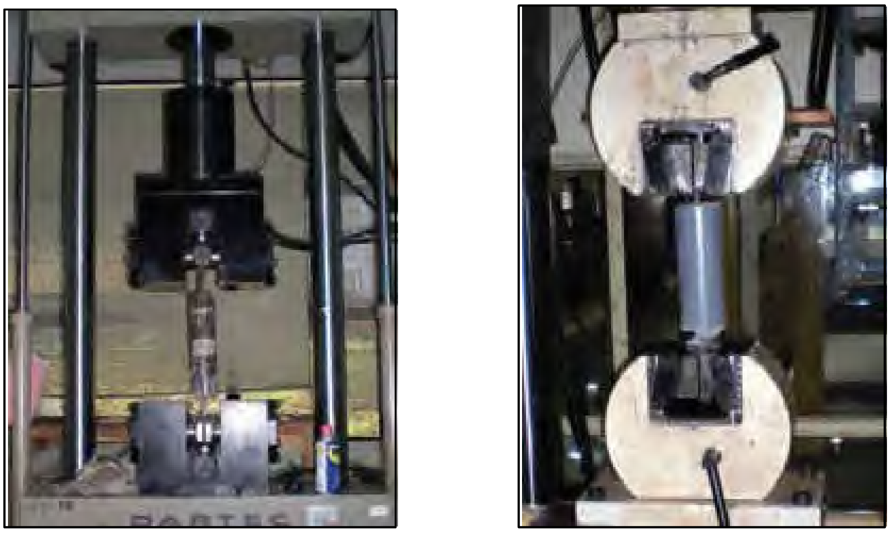
Specimen	Material for the sleeve	Length of sleeve (mm)	Diameter of sleeve (mm)	Effective thickness of sleeve (mm)	Grout strength f_{cu}, (N/mm^2)
A-Series	Mild steel pipe	200	50	4.05	43.32
B-Series	Corrugated aluminium sleeve	300	43	≈1.0	58.10
C-Series	Mild steel pipe	300	65	4.5	62.97
D-Series	Mild steel pipe	300	65	4.5 / -	51.47

Figure 5 shows the preparations and casting of the specimens. Wooden frames were used to hold the specimens in position before the grout was poured into the sleeve. The Y16 steel bars were inserted into the sleeve from both ends, meeting at mid length before they were tied to the frame. As the grout hardened and achieved the targeted strength of at least 40N/mm^2, the specimens were tested under incremental tensile load until failure (Figure 3), to obtain their ultimate loading capacity.

Figure 4. Tapered nut welded on steel bar

Figure 5. Casting of the specimens

Direct tensile tests were conducted in accordance with *BS EN 10002-1 2001 "Metallic materials – Tensile Testing – Part 1: Method of test at ambient temperature"*. Series A, B and C were tested using 250kN capacity actuator (Figure 6a). They were placed vertically and gripped to the actuator at about 11MPa pressure. Then, the actuator's arm moved upward gradually, inducing tensile force at a rate of 0.5kN/s. The relationship between incremental load and displacement were recorded during the test. As for D-series specimens, they were tested using the 1000kN capacity actuator (Figure 6b).



(a) 250kN capacity (b) 1000kN capacity

Figure 6. Direct tensile test using *Dartec* actuator

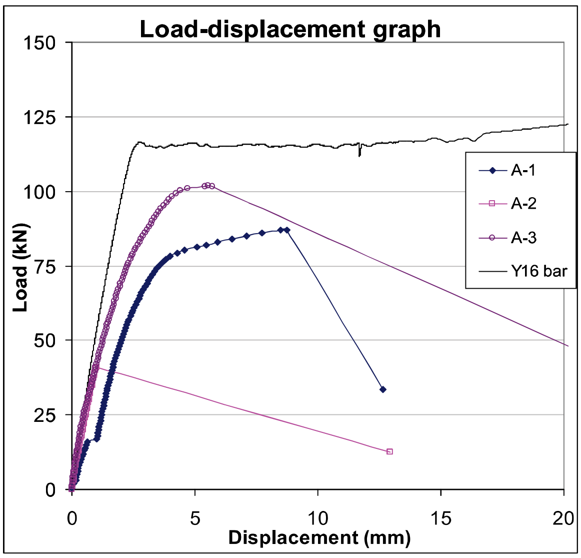
RESULTS AND DISCUSSION

Table 2 summarizes the tensile performance of the specimens, in terms of ultimate loading capacities, P (kN), the corresponding displacement, ΔL (mm), load at yield, P_y (kN), ductility ratios, $\Delta L/\Delta L_y$, and also the failure modes of each specimen. Figure 7 shows the relationship of load-displacement of the test specimens. The displacement readings obtained during the testing were the superimposition of; (a) elongation of steel bars, (b) elongation of splice sleeve, (c) bond-slip of the steel bars in the grout, and (d) bond-slip of the grout in the sleeve.

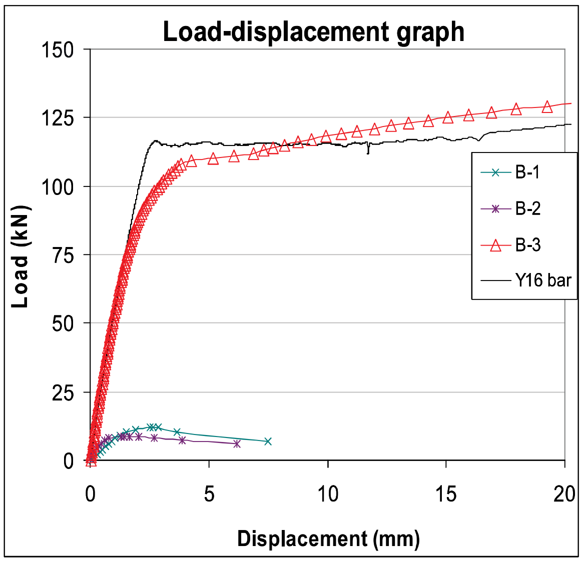
Table 2. Tensile performance of the specimens

Sample	Failure load P_u (kN)	Displacement ΔL (mm)	Load at yield P_y, (kN)	f_u/f_y	Displacement at yield, ΔLy (mm)	Ductility ($\Delta L/\Delta L_y$)	Failure mode
A-1	87.0	8.72	77	1.06	4	2.18	Bar slipped
A-2	40.9	1.00	40	0.50	0.8	1.24	Bar slipped
A-3	101.8	5.56	100	1.24	4.7	1.19	Bar slipped
B-1	11.9	2.65	10	0.14	1.9	1.40	Sleeve fractured
B-2	8.7	1.45	8	0.11	1.4	1.03	Sleeve fractured
B-3	135.6	35.51	108	1.65	3.4	10.44	Bar slipped
C-1	122.2	35.51	105	1.48	2.6	13.66	Bar fractured
C-2	134.8	25.45	116	1.64	4.4	5.78	Bar fractured
C-3	125.1	38.14	117	1.52	3.1	12.3	Bar fractured
C-4	135.4	29.48	117	1.64	3.2	9.21	Bar fractured
C-5	123.0	20.94	106	1.49	2.8	7.48	Bar fractured
D-1	96.0	27.19	76	1.17	7.6	3.58	Bar slipped
D-2	86.8	11.81	84	1.05	9.3	1.27	Bar slipped
D-3	94.9	13.94	90	1.15	9.8	1.42	Bar slipped
D-4	70.9	8.31	70	0.86	8.2	1.01	Bar slipped

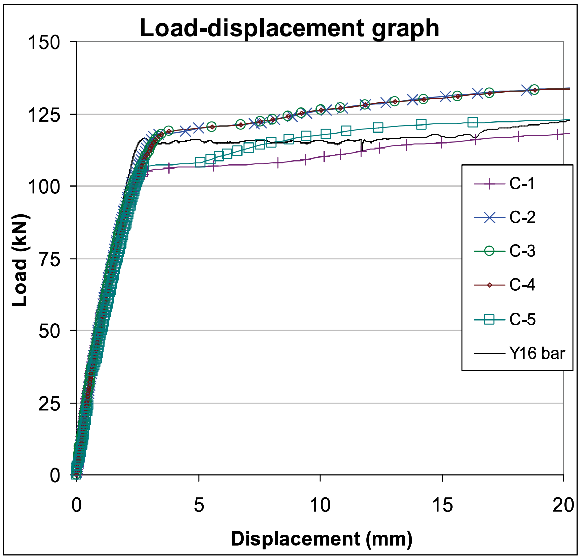
The performance of the splice sleeves was evaluated based on their ultimate tensile capacities, stiffness, yielding strengths, ductility and also failure modes. An adequate splice sleeve should have its bond strength equal or greater than the tensile capacity of the connected reinforcing bars. This is to (a) prevent splice failure before the connected reinforcing bars yield, for optimum usage of the capacity of precast elements, and (b) to ensure the ductile behaviour of the splice connection, for survival consideration. Thus, the test specimen will be considered adequate when the steel bars fractured outside the sleeve.



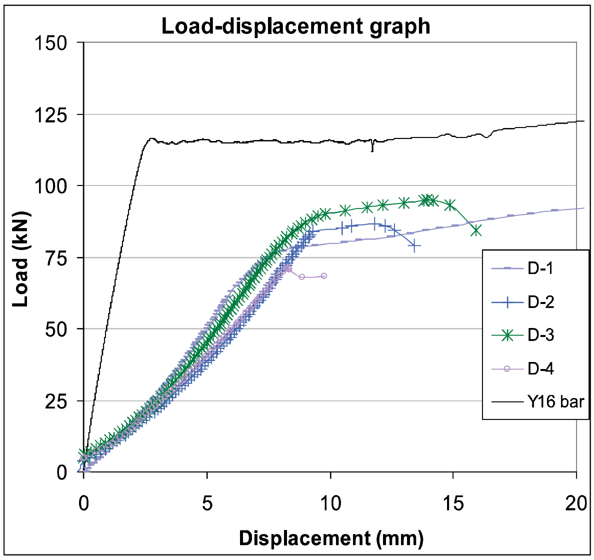
(a) A-Series



(b) B-Series



(c) C-Series

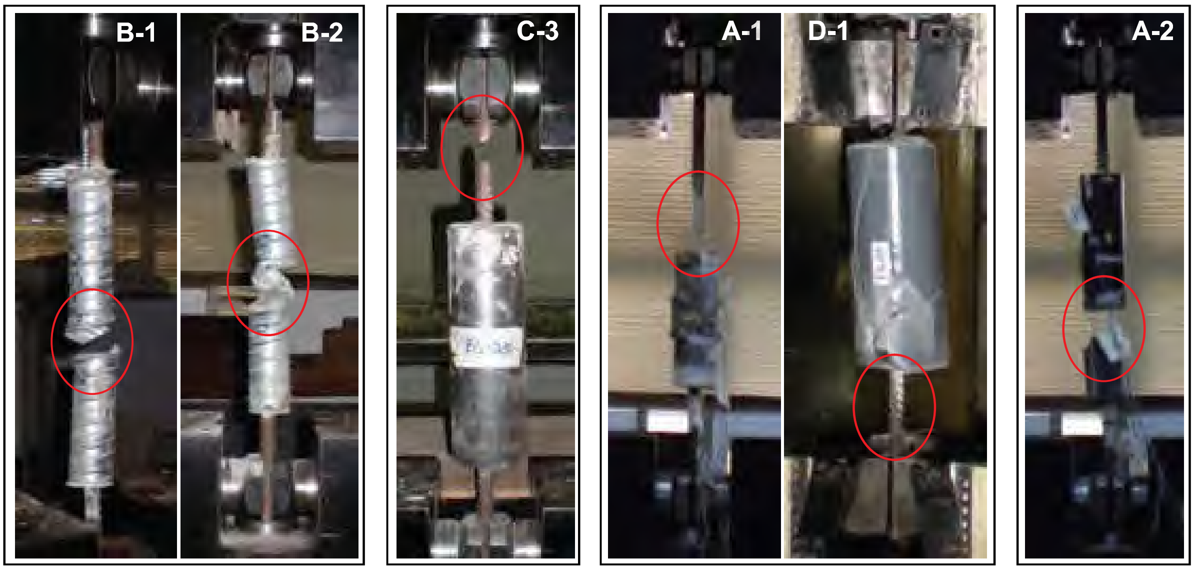


(d) D-Series

Figure 7. Load-displacement relationship up to 20mm

Failure modes

It is essential to study the failure modes, as they demonstrate the manner of defects and describe the causes of failure so that remedy and improvement can be made. Basically, the failure modes can be categorized into 4 types as shown in Figure 8. These include (a) sleeve fracture, (b) bar fracture, (c) bar slippage, and (d) grout slippage.



a. Sleeve fracture b. Bar fracture c. Bar slippage d. grout slippage

Figure 8. Classifications of failure modes

Sleeve fracture

B-1 and B-2 connectors, which utilized the corrugated aluminium sleeve as the sleeve body, fractured at mid length at the discontinuity of steel bars. The sleeve fractured as soon as the grout that bridged the reinforcement bars underwent tensile fracture. The corrugated aluminium sleeves were made of a long deformed aluminium sheet (approximately 1mm in thickness). They were rolled and attached together spirally with internal diameter of approximately 43mm, forming corrugated cylindrical sleeve to encourage grout-sleeve bond. At failure, together with the fracture of the grout, the aluminium sleeve of B-2 underwent tensile failure, while B-1 tore apart at mid length, approximately (see Figure 8(a)).

Bar fracture

All C-Series connectors had their spliced steel bars fractured outside the sleeve, indicating adequacy of bond strength. Either one of the upper or the lower bars, of which was relatively weaker in tension, fractured after enduring severe deformation. The malleable property of steel bar had caused the steel bar to reduce its cross sectional area as it elongated after yielding. This may be caused by higher stress concentration due to the smaller effective area. This occurred at several locations along the steel bars, where plastic deformation was observed. The failure mode of bar fracture was accompanied with a ductile load-displacement response that is preferable for construction, as shown in the C-Series curves of Figure 7.

Bar slippage

A-Series, C-Series and B-3 connectors failed as their steel bars slipped from the surrounding grout. Before the slippage, it was expected that the bar-grout bond relies mainly on the mechanical interlocking effect between the bar ribs and the grout keys, with minor contribution by friction and chemical adhesion, as observed by Lutz (1966) and Goto (1971). In this case, the interlocking mechanism relied on the shear area of the grout keys to resist the slipping force that was derived from the resultant acting perpendicularly to the inclined rib surface (see Figure 9a). The distribution of bond stress along the embedded length is illustrated in Figure 10, which is modified from the bond stress distribution diagram by Ferguson (1988). Initially, at small tension force, higher stress was found near the ends of the bars where the loading was applied, engaging the grout keys at the region to resist the force. Slip occurred as the adhesion between the bar and the grout in this region broke down and the bar ribs began to crush some parts of the grout keys. Then, further loading increment had shifted the bond stress deeper along the bars from both ends of the sleeves, engaging additional ribs to resist the load. The slip of steel bars accumulated as the load increased. Then, as the incremental load was approaching the bond limit, shifting of the bond stress continued as bond stress peaks moved deeper along the bar. Eventually, this triggered ultimate slip of steel bar, because there was no capacity left provided by the interlocking of the steel ribs and the concrete. For most of the A-Series and D-Series specimens, the slip failure occurred suddenly. This failure mode is not preferable in construction, as it indicates that the connectors lack the characteristic of ductility.

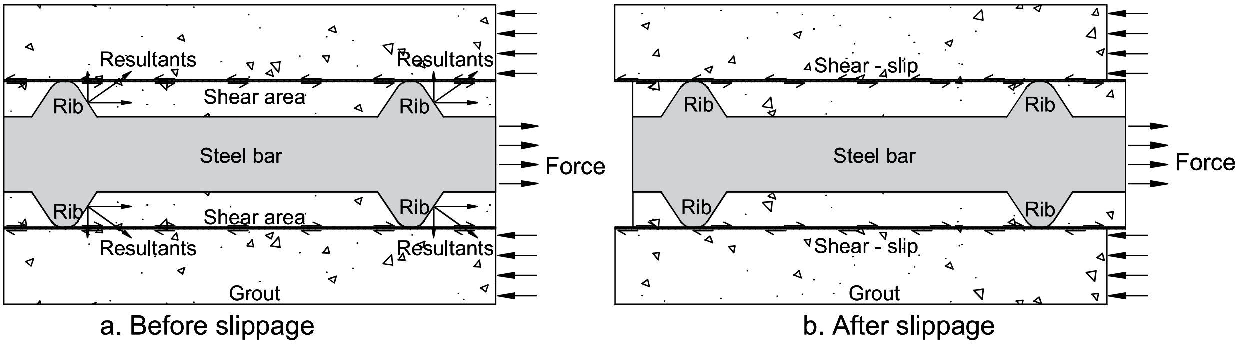


Figure 9. Slippage Mechanism

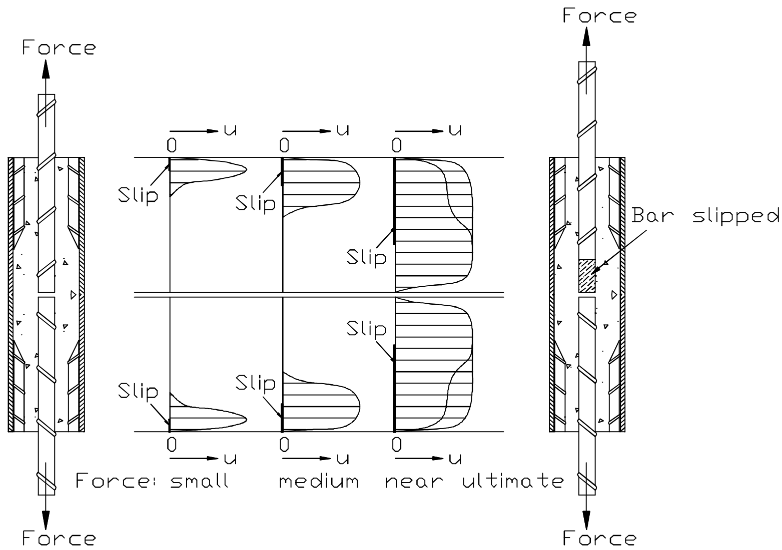


Figure 10. Bond Stress Distribution (modified from Ferguson et al., 1988)

Grout slippage

Grout slippage is defined as the slippage occurred between the grout and the sleeve. The typical failure mode of the grout slippage is shown in Figure 8d. As seen in specimen A-2, it failed as the grout slipped together with the reinforcement bar. It was due to the inexistence of the interlocking mechanism between the steel pipe (sleeve) and the grout. The tensile resistance relied on the surface friction between the grout and the steel pipe, of which, according to the theory of friction, increases proportionally as the splitting force (the component force derived from the resultants acting perpendicularly to the inclined surface of bar ribs) acting on the sleeve wall increases (Figure 11). Eventually, at ultimate load, the grout fractured at the discontinuity of the steel bars and subsequently slipped at 40.9kN load.

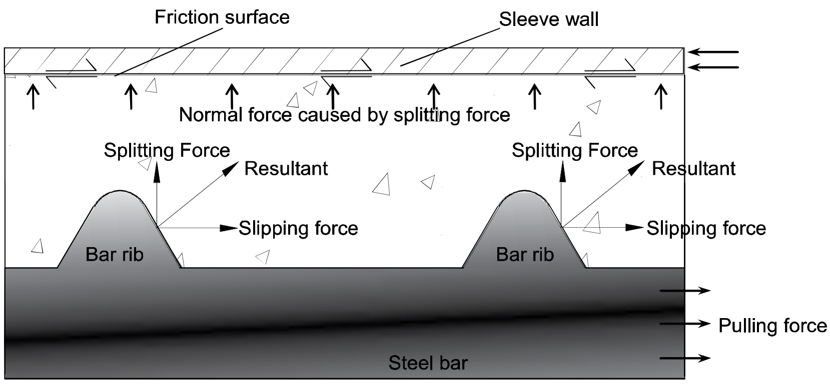


Figure 11. Friction-slip mechanism of A-2

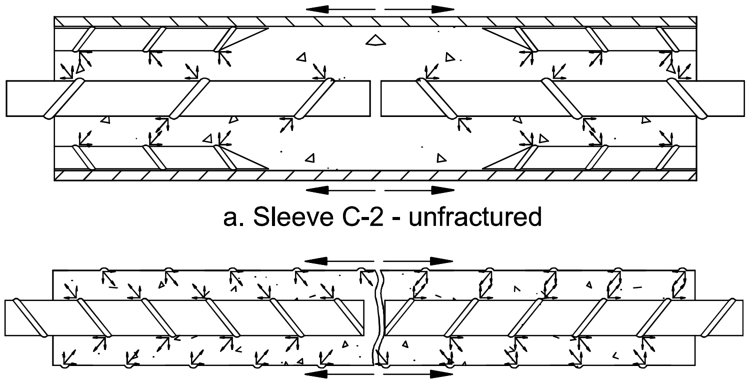
Tensile Resistance

The tensile resistance of each connector is defined as the maximum load that can be carried and is indicated by the highest failure load obtained from the tensile tests. In this study, the tensile resistance of the connectors ranges between 8.7kN and135.6kN. Based on the failure modes, the tensile capacities of these connectors are governed by; (a) the tensile strength of the sleeve, (b) the rebar-grout bond strength (c) the grout-sleeve bond strength, and (d) the ultimate tensile strength of the connected reinforcement bars. These factors are discussed in details as follows.

Tensile strength of sleeve

The results show that the sleeve made from mild steel pipe (A and C-series) provides higher tensile resistance compared to the corrugated aluminium sleeve (B-1 and B-2). In the case of the mild steel pipe, the designed yield strength was 250N/mm^2, and the thickness of the pipes was at least 4mm. The tensile resistance of the splice sleeve can be acquired by multiplying the yield strength with the effective cross sectional area of the sleeve. In this case, it was about twice higher than the tensile capacity of the connected reinforcing bars and as a result A and C series remained unfractured at ultimate load (Figure 12a).

As for B-1 and B-2 series, the end-to-end arrangement of steel bars had caused the aluminium sleeve and the grout that bridged the discontinuity of the steel bars to carry the tensile load (Figure 12b). It is fairly assumed that the tensile strength of the grout was approximately 10% of its compressive strength, which was 5.81N/mm^2. Therefore, by multiplying the tensile resistance stress with the effective grout area (sleeve area - bar area), the tensile resistance of the grout in the sleeve was 6.9kN. As for the corrugated aluminium sleeves, only limited effective area took part in the tensile resistance since the thickness was approximately 1mm. The tensile stress of the aluminium sleeve was only 90N/mm^2, which was 36% of the mild steel metal. Multiplication of 90N/mm^2 with the effective cross sectional area is equal to 6.0kN. The combination of the tensile resistance contributed by the grout and the aluminium sleeve equals to 12.9kN, which is fairly reasonable as compared to the 11.9kN ultimate load of specimen B-1, of which both the grout and the aluminium sleeve fractured at ultimate.



a. Sleeve C-2 - unfractured

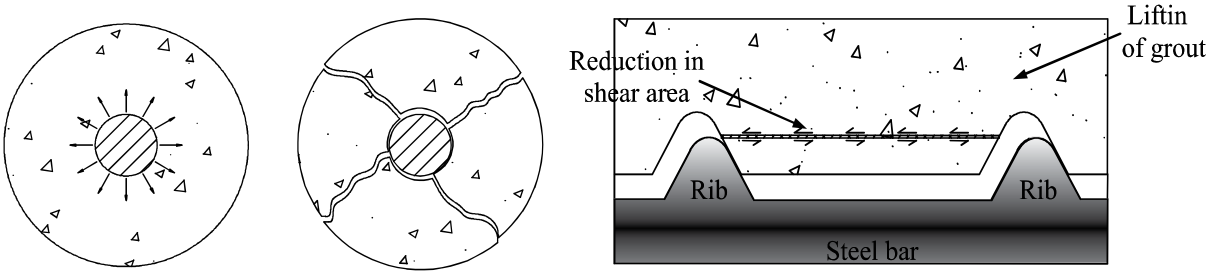
b. Sleeve A-2 fractured

Figure 12. Load distribution diagram

Tensile strength contributed by the bar-grout anchorage bond

It is known from the literature review that the anchorage length of the steel bar, the compressive strength of the bonding material, and the degree of confinement influence the the bar-to-concrete bond performance (Soroushian et al., 1991, Hayashi et al., 1993, Amin Einea et al., 1995). In this study, the influence of these factors was rather not obvious and not measurable. This is because the embedded length and the grout strength were not varied, and also the degree of confinement in the sleeve was rather difficult to quantify.

However, it can be seen that, the higher bond strength of C-Series was partially contributed by higher grout strength of 62.97N/mm^2 compared with the other specimens as listed in Table 1. Besides that, the confinement effect also contributed to the higher bond performance in the sleeves. For A-2, C-3 and C-4 series, the confinement forces were generated due to peripheral tensile resistance of the sleeves, responding to the splitting forces derived from the resultant of the mechanical interlocking mechanism between the Y16 bar ribs and the grout keys. These passive confinement forces resisted the splitting failure of the grout that may decrease the effective shear area of the grout keys in resisting the pull-out force (Figure 13).



a. Splitting force b. Splitting failure c. grout keys shear area

Figure 13. Degradation of bond resistance due to splitting cracks

A similar principle applies in B-3, where the steel bars were spliced adjacently throughout the length of the corrugated aluminium sleeve. Connector B-3 presented higher bond strength of 135.6kN, although it eventually slipped after a certain degree of elongation.

The 300mm confined lap length is approximately 46% shorter than the required conventional lap length by BS-8110 (which is 35 x bar diameter = 560mm).

C-1 and C-2 provides more efficient confinement to the spliced bars. They generated additional confinement forces instead of relying solely on the passive forces that control the splitting cracks. The active confinement forces were generated by the Y10 bars that were welded onto the sleeve upon movement of the grout together with the Y16 bar towards the direction of the pulling force. These confinement forces were derived from the resultant acting perpendicularly to the Y10 bar ribs and towards the centroid of the sleeve (Figure 14). These confinement forces had efficiently enhanced the bar-grout bond performance by (a) controlling the splitting cracks that leads to the degradation of bonding strength, and (b) inducing additional normal forces to reinforcement bar to increase the friction resistance with the grout.

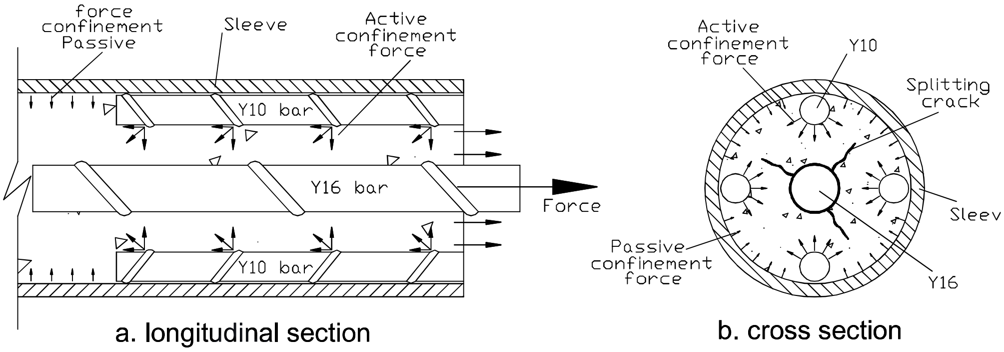


Figure 14. Confinement forces generated in C-1 and C-2 connectors

The comparison between C and D-series proved that confinement should be provided throughout the splice length in order to acquire more efficient bond performance. However, the provision of the confinement was inconsistence throughout the length of D-series. Only several portions of the grout (50mm at mid length and 25mm from both ends of the sleeve) were confined using steel pipes. Although D-Series were cast and tested in Polyvinyl Chloride (PVC) pipe, its confinement effects was assumed negligible because of (a) large diameter (110mm) which led to less significant confinement effect to the bond performance, and (b) its plasticity behaviour, which is hardly predictable in the study. The result shows that the D-series provided an averagely about 31% lower tensile capacities compared to the C-Series.

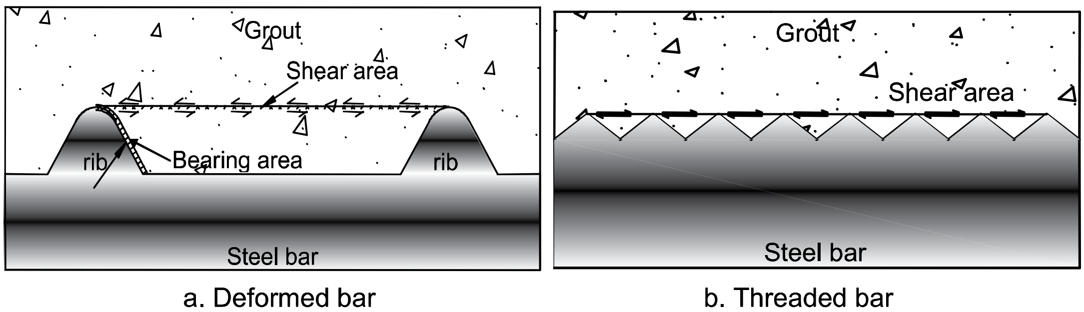


Figure 15. Bearing area versus shear area on reinforcement bars

Besides that, the surface conditions of the steel bars, which include the rib patterns and the distance between bar ribs, affect the bar-grout bond performance. This is evidence from specimens A-3 and A-1 that utilized deformed and threaded steel bars respectively. The tensile capacity of A-3 was 14.8% higher as compared to A-1. This result was contributed by the 0.1 relative rib areas of the deformed steel bars used in A-3, which was recognized by many researches as the optimum rib design, due to the optimization between the bond performance and cost-benefit (Abram et al., 1913, Clark et al., 1946, 1949, Darwin et al., 1993, Hamad et al., 1995). The deformed steel bar in A-3 provides higher bond strength compared to the threaded steel bar in A-1. This higher bond strength was contributed by the higher shear/bearing area, thus, engaging larger shear area to resist the slipping force as compared to the threaded steel bars (Figure 15). Since the bond strength was not equally distributed along the embedded length (Goto et al., 1971, Mains et al., 1951), higher relative shear area would provide better resistance towards progressive failure of the grout keys. Hence, the deformed steel bars provide better bond performance compared with the threaded steel bars.

Tensile strength contributed by the grout-sleeve anchorage bond

The grout-sleeve bond is an essential requirement to resist the grout from slipping out of the sleeve. Specimen A-2 failed at 40.9kN due to inexistence of interlocking mechanism between the grout and the sleeve. However, by welding the steel plate on the semi-circular pipes as in specimens A-1 and A-3, or by welding Y10 bars that have deformed ribs as in C-1 and C-2, or even by providing welded rings onto the sleeve as in C-3, C-4 and C-5, improved the tensile performance drastically. Figure 12 shows the grout-sleeve bond mechanism for different configurations of the sleeves. Specimen A-2 relied on the surface friction resistance between the grout and the sleeve. Specimen A-3 had an additional sleeve end resistance despite of relying solely to the friction-slip resistance. In C-2 specimen, the grout-sleeve bond was contributed by the surface friction-slip resistance and also the interlock-slip mechanism to the Y10 bars on the sleeve. Finally, C-5 relied on the effective shear area on the grout to resist the slipping force.

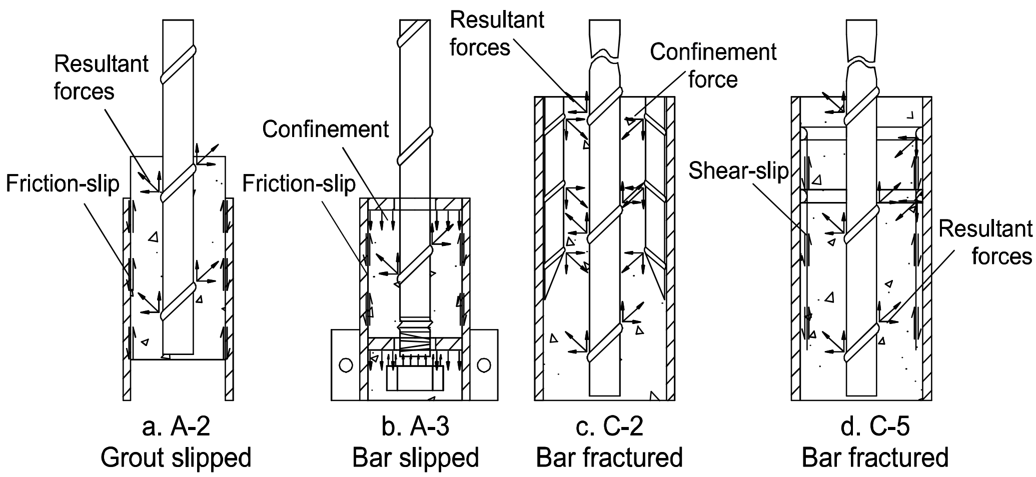


Figure 12. Grout-sleeve bond mechanisms comparison among specimens

Tensile strength of steel bars

All C-Series specimens had their steel bars fractured outside the sleeve, with the fracture loads ranging from 122.2kN to 135.5kN (equivalent to 607.7N/mm^2 and

672.4N/mm^2 of bar fracture stress). Excellent bond performance was generated in the sleeves. Although the fractured strengths were inconsistent, the quality of the steel bars were still reliable, as they fractured beyond the yield strength of 410N/mm^2, of which was 48% to 64% higher. In common practices, the reinforcement bars in reinforced concrete are designed to endure stress less than yielding stress of 410N/mm^2 (Y-type high strength steel bars), to ensure continuous elastic behaviour of the global structural system for maximum performance. The C-series specimens produced much higher bonding than the yield strength. Therefore, the C-series connectors are suitable to be used in construction industry, provided that sufficient grout strength is provided.

Stiffness

Stiffness is the ratio of the tensile load divided by the corresponding displacement. The specimen with high stiffness ratio endures relatively smaller displacement with correspondence to large tensile force. In load-displacement curve, the stiffness is represented by the slope of the curves.

Due to excellent bar-grout bond, specimens C-1, C-2, C-3, C-4 and C-5 presented higher stiffness as compared to others (interpreted from the slope of load-displacement curve of Figure 4). Then, the stiffness decreased drastically, followed by severe elongation of steel bars before they eventually fractured at 122.2kN, 134.8kN, 125.1kN, 135.4kN, 123.0kN load, accordingly. On the other hand, specimens D-1, D-2, D-3 and D-4 presented relatively lower stiffness due to the (a) inconsistence confinement along the splice length of the specimens, where only 50mm at mid length and 25mm at both ends were confined by using mild steel pipes.

Yield Strength

The yield strength is identified as the steel begins to behave plastically, where some fraction of deformation becomes permanent and non-reversible. The yield strength of the steel is commonly determined by finding the intersection of the load-displacement curve with the parallel 0.2% offset strain line to the initial slope of the curve. However, in this case, the tensile resistance of the splice sleeves is governed by the composite action between the steel bars, the grout and the sleeve. Perfect elastic behaviour is impossible, as the stiffness of the specimens degrades progressively throughout the testing due to propagation of minor internal cracks as a result of interlocking reaction between bar ribs and the grout keys (Figure 14). Therefore, the yield strength obtained from the offset strain line method may not be representative to indicate the significant decrease in stiffness or the beginning of the plastic behaviour. Thus, the yield strengths of the splice sleeves are estimated upon the occurrence of drastic decrease in stiffness (the slope of load-displacement curve).

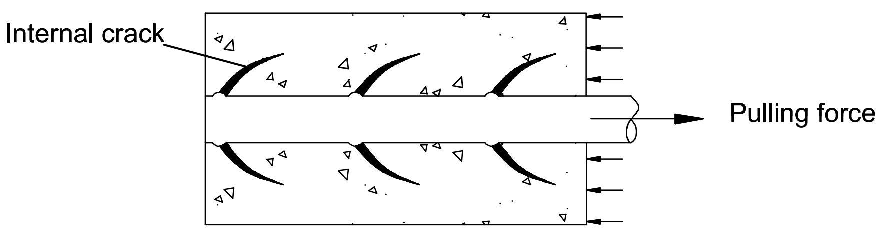


Figure 14. Internal crack propagation in grout (modified from Yankelevsky (1985))

The yield strength of the specimens, approximated from the load-displacement curves of the specimens, ranged from 8.4kN and 118.8kN. The occurrence of sudden decrease in stiffness of the specimens was governed by (a) bar-grout bond capacity, in the case of specimens A-1, A-3, B-3, D-1, D-2, D-3 and D-4, (b) grout-sleeve bond capacity, in the case of A-2, (c) sleeve tensile capacity, in the case of B-1 and B-2, and also (d) bar yielding strength, of which was weaker.

Ductility

Ductility is the property of steel that permits it to reduce in the cross sectional area under tension, before it fails ultimately. An adequate splice sleeve to be utilized as connections in precast concrete structures should provide ductile behaviour. It should endure considerable elongation before it eventually fails by necking, with consequent rapid increase in local stresses. In this study, the ductility ratio is obtained by dividing the corresponding displacement at ultimate state (ΔL) with the initial yield displacement (ΔL_y) of the specimen. The specimens with low ductility ratio (A-series, C- series, B-1 and B-2) are not preferable for use in construction, because they may experience sudden failure.

The grout-filled splice sleeve is formed with two different materials of different behaviour. Slippage of bars from high strength grout gives brittle failure, while fracture of high strength steel bars gives ductile failure. In order to obtain ductile behaviour, the bond should not fail before the steel bars yield, and the steel bars should undergo a certain degree of elongations before fracture. The C-Series presented satisfactory results as they fulfilled the requirements. In terms of loading behaviour (interpreted from the load-displacement curves in Figure 4), they also follow closely to the typical load-displacement pattern of a steel material, enduring through the stages of elasticity, strain-hardening, rupture and also necking.

As for specimen B-3, the reinforcement bars slipped out of the sleeve before reaching the ultimate tensile capacity. Judging on the failure mode, it may not be an adequate splice sleeve requirement. However, the fractured load of 135.6kN is about the capacity of a Y16 steel bar. It also presented the ductility ratio of 10.44, signifying satisfactory elongation before reaching the ultimate state. Hence, B-1 may also be acceptable for construction.

CONCLUSION

In this study, a total of fifteen proposed splice sleeves were experimentally tested under incremental static tensile load and were evaluated for feasibilities, based on their stiffness, yielding strength, ultimate tensile capacities, ductility ratios and failure modes. The study concluded that:

i) An adequate splice sleeve connectors should have the following criteria:
   a. undergoes failure mode of bar fractured instead of slippage
   b. presents high stiffness behaviour, where small displacement with correspondence of large pulling force takes place

c. presents high bond strength, which is comparable to the tensile resistance of the spliced steel bars, and also high ductility ratio for survival considerations in real practice.

ii) Based on the failure modes observed, the tensile capacities of a splice sleeve are governed by the following factors which are to be considered in the design of splice sleeve:
   a. tensile strength of the sleeve,
   b. the rebar-grout bond strength,
   c. the grout-sleeve bond strength, and
   d. the tensile strength of reinforcement bars.

iii) Specimens C-1, C-2, C-3, C-4 and C-5 are considered acceptable and feasible to function as a connection system to carry full tensile load.

iv) As for specimen B-3, although the connected reinforcement bars slipped, it produced high bonding strength and high ductility ratio. Thus, it is also considered acceptable.

ACKNOWLDEGEMENT

The authors gratefully thank the CREAM (Construction Industry Research Institute of Malaysia) and CIDB (Construction Industry Development Board) for the financial support under research grant Vot. 73713. The experimental programs were conducted at the Structural and Material Laboratory of Faculty of Civil Engineering, Universiti Teknologi Malaysia, Skudai. The contribution of the technical staff is also acknowledged

REFERENCE

Amin Einea, Takashi Yamane, Maher K. Tadros, (1995), Grout-Filled Pipe Splices for Precast Concrete Construction, *PCI Journal*, Vol. 40(1):82-93

BS8110-1:1997 (1997). Structural Use of Concrete - Part 1: Code of practice for design and construction, *British Standards BSI*

BS EN 10002-1 2001, (2001), Metallic materials – Tensile Testing – Part 1: Method of test at ambient temperature, *British Standards BSI.*

Clark, A. P. (1946). Comparative Bond efficiency of Deformed Concrete Reinforcing Bars. Proceeding of, Detroit, Michigan, ACI Journal.

Clark, A. P. (1949). Bond of Concrete Reinforcing Bars. Proceeding of, Detroit, Michigan, *ACI Journal*

Coogler, K. L., (2006) *Investigation of the Behaviour of Offset Mechanical Splices.* Master of Science Thesis, Department of Civil and Environmental Engineering, University of Pittsburgh, 118 pp.

Darwin, D., Graham, E.K, , (1993), Effect of Deformation Height and Spacing on Bond Strength of Reinforcing Bars, *ACI Journal*, Vol. 90, 6, pp

Ferguson, P. M., Breen, J. E., and Jirsa, J. O. (1988). *Reinforced Concrete Fundamentals.* New York, NY., John Wiley and Sons, Inc.

Goto, Y., (1971), Cracks Formed in Concrete Around Deformed Tension Bars, *ACI Journal*, Vol. 68(4):244-251

Hamad, B. S., (1995), Comparative Bond Strength of Coated and Uncoated Bars with Different Rib Geometries, *ACI Material Journal,* Vol. 92, No 6

Hayashi, Y., Shimizu, R., Nakatsuka, T., Abd Suzuki, K., (1993), Bond Stress-Slip Characteristic of Reinforcing Bars in Grout-Filled Couplin Steel Sleeve*, Proceedings, Japan Concrete Institute*, Vol. 15, No.2, pp 265-270

Ling Jen Hua, Ahmad Baharuddin Abd. Rahman, Abdul Karim Mirasa, Zuhairi Abd. Hamid, (2008), Performance of CS-Sleeve Under Direct Tensile Load: Part I - Failure Modes*, Malaysian Journal of Civil Engineering*, Vol. 20(1):89-106

Ling Jen Hua, Ahmad Baharuddin Abd. Rahman, Abdul Karim Mirasa, Zuhairi Abd. Hamid, (2008), Performance of CS-Sleeve Under Direct Tensile Load: Part II - Structural Performance *Malaysian Journal of Civil Engineering*, Vol. 20(1):107-127

Ling Jen Hua, Ahmad Baharuddin Abd. Rahman, Zuhairi Abd. Hamid (2008). Failure modes of Aluminium Sleeve Under Direct Tensile Load. *Proceeding of 3rd International Conference on Postgraduate Education (ICPE-3)*, Penang, Sarawak Universiti Sains Malaysia (USM)

Ling Jen Hua, Ahmad Baharuddin Abd. Rahman, Zuhairi Abd. Hamid (2008). Performance of Corrugated Aluminium Sleeve Connector under Direct Tensile Load *Proceeding of 2nd Engineering Conference on Sustainable Engineering (ENCON 08)*, Kuching, Sarawak, Malaysia, University Malaysia Sarawak (UNIMAS)

Lutz, L. A., (1966), The Mechanics of Bond and Slip of Deformed Reinforcing Bars in Concrete Department of Structural Engineering, Cornell University New York

Thompson M. K., Breen J. E. and Klingner R. E. (2002), Anchorage Behaviour of Headed Reinforcement: Literature Review, *Research Report 1855-1*, University of Texas, Austin

Mains, R. M. (1951). Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars *Proceeding of*, Detroit, Michigan, *ACI Journal*

Nilson, A. H., (1975), Internal Measurement of Bond Slip, *ACI Journal*

Peter O. Jansson, (2008), Evaluation of Grout-Filled Mechanical Splices for Precast Concrete Construction, *Research Report R-1512*, Michigan Department of Transportation MDOT, Michigan

Sika Grout®-215, Product Date Sheet, Edition 0808/ 1, *Sika Kimia Sdn Bhd*, Malaysia

Soroushian, P., Choi, K., , (1989), Local Bond of Deformed Bars with Different Diameters in Confined Concrete *ACI Structural Journal* Vol. 86(2):217-222

Soroushian, P., Ki-Bong Choi, Gill-Hyun Park, Farhang Aslani, (1991), Bond of Deformed Bars to Concrete: Effects of Confinement and Strength of Concrete*, ACI Material Journal*, Vol., pp 227-232

Splice Sleeve North America, Inc., (2007), *NMB Splice Sleeve System User's Manual*, Irvine, CA

Untrauer, R. E., Henry, R. L., (1965), Influence of Normal Pressure on Bond Strength*, ACI Journal* Vol. 65(5):577-585

Yankelevsky, D. Z., (1985), Bond Action between Concrete and a Deformed Bar - A New Model, *Proceeding of ACI Journal*, Vol. 82(2):154-161

Malaysia Construction Research Jurnal; **Vol.6**|No. 1|2010

EXPLORATORY STUDY ON WOMEN CONSTRUCTION ENTREPRENEURS IN MALAYSIA

Mastura Jaafar[1], Abdul Rashid Abdul Aziz and Zulaiyahaa Abdul Rahman
[1] *School of Housing, Building and Planning, Universiti Sains Malaysia, 11800 Penang, Malaysia.*

Abstract
Starting a business in a male-dominated industry is a challenging task for woman entrepreneurs. Hence, this paper aims to assess women involvement in the construction industry. Fifty survey questionnaires were sent to women contractors in the northern region of Peninsular Malaysia i.e., Penang, Kedah, Perlis, and Perak. The findings revealed that the majority started with small businesses, which for some of the respondents successfully grew. These women operated as full time contractors and highly emphasised the need to work hard. Most of them gained business knowledge through formal courses and received assistance from the government. Their experience and industry knowledge helped in terms of managing their firms and finding external opportunities. The research concludes that it is not impossible for women to achieve success in a typically male-dominated sector. To capture the general scenario of women's involvement in the construction industry, this study proposes a thorough research to be conducted in Malaysia.

Keywords: *Women construction entrepreneurs, construction industry, business*

INTRODUCTION

Traditionally, the construction industry is a demonstrably male-dominated economic activity (Gale, 1991 and 1992 and Gale and Cartwright, 1995). It was recorded in 1981 that women constituted less than 7% of the full-time workforce in the UK construction industry and of these, over 80% were employed in traditional 'women's jobs', such as secretarial and clerical (Gale, 1991). However the percentage of women workforce in the UK and within the EU countries has increased substantially (Gale and Susan, 1995). In Singapore, Teo (1996) found that the majority of female business owners seem to be in the service and retailing industries with very few in the construction line. However, Vervey (2007) indicated that there is a current trend of women becoming interested in exploring activities dominated by males. According to Ahl (2006), studies on women entrepreneurs suffer from a number of shortcomings, among which is the lack of recent studies, as most were conducted in the 1990s and focused on non-construction industries, such as the studies of Teo (1996) and Maysami and Goby (1999).

In Malaysia, records show that the number of women involvement in business increased by about 38% in the five-year period beginning 1990 (Statistical Department Malaysia, 1999). According to Abdullah (1999), one of the important factors contributing to this is the increasing involvement of women in the manufacturing and services sectors. Contemporary women have been found to be keen to move away from sectors such as education, health care, and office work and towards those traditionally dominated by men (Karim, 1997). Maimunah (1996) found that women in Malaysia became involved in various enterprises formerly dominated by men. According to data from the Malaysian Construction Industry Development Board (CIDB), currently there are almost 65,000 contracting firms in the industry. Nevertheless, the actual number of women entrepreneurs in the industry is unavailable. Jaafar (2003) supported the findings of studies in other countries (e.g., Holmqlist and Sudin, 1984) that indicate women entrepreneurs' involvement in the construction industry is still low.

The establishment of the Ministry of Entrepreneurial and Cooperative Development (MECD) in 1995 and the Ministry of Women, Family, and Community Development (MWFCD) in 2001 indicates the strong government support in the development of women entrepreneurs. These ministries, together with other supporting government agencies, provide support to women entrepreneurs in terms of funding, physical infrastructure, and business advisory services (Teoh and Chong, 2007). For example between August 1999 and March 2006, 536 women-owned companies were awarded grants and soft loans by the Small and Medium Industries Development Corporation (SMIDEC) worth RM51.5 million. Furthermore, many women entrepreneurs associations have been established to promote and encourage more Malaysian women to become entrepreneurs.

Particularly in the construction industry, women are significantly underrepresented (Equal Opportunities Commission [EOC], 1990). Poor public image of the construction industry such as dangerous, dirty, difficult, and demanding have restricted women involvement in the industry. Gale and Susan (1995) used keywords such as crisis, aggression and conflict, gallant behaviour, and traditional attitudes to explain the culture of the industry that is related to male domination. They found that women in construction have to tolerate overt sexist language and behaviour, which women in other occupations do not accept.

Contractor is not considered a professional career compared with other occupations in the industry. Gale (1994) viewed the industry as having a few sub-industry cultures according to size and output of the firms. There are various specialisations in the construction industry more suited to woman, such as the consultancy business in the fields of engineering, planning, and surveying. Each of these professional practices has its own cultural and image characteristics (Gale, 1994). The consultancy business, which is more associated with indoor activity, is more comfortable and suitable for women compared with construction.

Construction firms can be categorised using their CIDB grades, that is, G1-G7. These grades are determined by their paid up capital and tendering capacity. Tendering capacity in turn can be used to classify project size as follows: micro project (G1) with a value less than RM200,000, small project valued between RM200,000 and RM1 million, medium project worth between RM1 million and 5 million, and large project valued above RM5 million. For every project development, project implementation can be considered a very critical stage. The contractor is given a specific time frame to complete the building using the specifications given at an agreed upon price. With the increasing practice of sub-contracting, there is a very important skill that should be acquired by a contractor: the ability to manage sub-contractors and site workers. Many researchers for examples Jaafar (2003) agree that to succeed in the industry, contractors have to excel in managing people. Furthermore, in supply chain management, the contractor acts as a hub in managing the relationship among the client, consultant, supplier, and sub-contractor.

In view of the above discussion, this study was conducted to explore women construction entrepreneurs in Perak, Kedah, Perlis and Penang. As the preliminary study of a larger research, the data presented are based on the perceptions of 14 respondents from the industry. This paper discusses the respondents' background and their firms' characteristics, as well as the challenges and problems women entrepreneurs encounter.

LITERATURE REVIEW

The word entrepreneur conveys different meanings to different people. For some, an entrepreneur is an individual who has a unique personality not possessed by normal individuals. Mamat et al. (1990) proposed that an entrepreneur undertakes a venture, organises it, raises capital to finance it, and assumes all or a major portion of the risks. In the Malaysian census, women entrepreneurs are classified under the employment classification of employer, self employed or own-account workers, and unpaid family helpers (Teoh and Chong, 2007). Feather (1988) referred to successful owner/managers as those who have entrepreneurial characteristics. In relation to the construction industry, Van Der Merve (2003), defined a successful woman entrepreneur as someone who has been in the business for longer than two years, operates an enterprise with more than five employees, makes a profit, and able to expand her business in terms of infrastructure and growth.

Demographic profile

Factors that motivate a woman to become an entrepreneur can be explained by both the push and pull factors. Watkin and Watkin (1982) argued that parent involvement in business could influence daughters to become entrepreneurs. Robert et al. (1981) proved that 60% of women in the manufacturing sector and 80% of women in the construction sector have entrepreneurs as parents. There are other family-related factors. Hisrich and O'Brien (1981) and Watkin and Watkin (1982) found that the majority of woman entrepreneurs are first-born children. Thus, their career decisions can be explained by their role in carrying greater family responsibilities, making them more independent. Several studies in other countries found that the majority of female business owners are aged between 30 and 51 years old, married, and with only a few children (Deng et al., 1995; Teo, 1996, and Maysami and Goby, 1999).

Education level is also influential. According to Watkin and Watkin (1982), an entrepreneur chooses his/her business based on acquired knowledge and formal educational level. A few studies like that of Teo (1996) found that female business owners are generally well-educated, with the majority having at least a secondary level of education. Dolinski et al. (1993) supported the importance of education and suggested that less educated women might face financial or human capital constraints that limit their business pursuits.

Moreover, experience in business is important to explore various opportunities in the business sectors (Vester, 1980). Another important motivation is access to funding. Most entrepreneurs start their businesses with limited capital, usually their personal savings. In Australia, Deng et al. (1995) found that small business owners prefer to locate their businesses at home to reduce unnecessary overhead costs.

Other than demographic variables, successful business owners have been identified to possess a personal value type, which is referred to as entrepreneurial (Cunningham and Lischeron, 1991). It has also been argued that entrepreneurs place high value on ambition, achievement, reliability, responsibility, hard work, competence, optimism, innovation, aggressiveness, honesty, creativity, social recognition, and growth (Kotey and Meredith, 1997). Brush and Hisrich (1991), conducted a study on women entrepreneurs, and they agreed on the significant impact of managerial skills and particular strength in generating ideas and dealing with people on business performance. According to Buttner (2001), the management styles of women entrepreneurs can be best described as using relational dimensions such as mutual empowerment, collaboration, sharing of information, empathy, and nurturing, which are associated with business performance. Shim and Eastlick (1998) agreed with the importance of good networking on the survival of female-run establishment.

A study on small and medium business owner/managers in the Malaysian construction industry found that contractors demonstrate a high pursuit of excellence and work ethics that motivates them to work hard and complete their work accordingly (Jaafar, 2003). Work ethics is related to the contractor's aspiration to work hard in his/her business. In the construction industry, competition is intense due to the ease of entry. Thus, contractors have to work hard to secure construction orders. The same argument is given by Vervey (2007) who recommended that women entrepreneurs in construction should build and develop their environment and economy, starting with low a profit margin which is a common strategy for survival coupled with the drive to meet challenges, overcome barriers, and nurture their businesses.

Individual motivation has been found to be related to performance in women-owned businesses (Brush, 1990). Nordin (2005) revealed that among other factors, psychological motives, such as self-satisfaction and source of finance affect women entrepreneurs. The importance of strategy for business survival has been discussed by researchers since the 1960s. Strategy ensures the fit between changes in the environment and the entrepreneur's ability to develop and execute more effective strategy. Bracker et al. (1988) found that firms which undertake strategic planning perform better financially.

Challenges and problems

Women entrepreneurs face various challenges. However, Smeltzer and Fann (1989) did not find any difference in business performance between male- and female-owned businesses in terms of number of employees, annual revenues, or growth rates. However, Watson (2002) argued that female-owned businesses generally underperform compared with male-owned businesses, although he did point out that the results might differ between industries. Among the reasons cited by researchers are age of business, family commitment, accessibility to capital, level of education and prior experience, level of risk aversion, and level of concern on financial rewards.

Other than that, the most challenging task for women is to balance out the traditional tasks of being a wife and mother with their careers (Kamaruddin, 1997). Kartz (1989) argued the possibility of women taking fewer risks. Stone et al. (1990) highlighted the work-home conflict that arises from the dual responsibility of managing a business and maintaining a family. This conflict is likely to be a main stumbling block for female business owners.

One of the most important problems of women entrepreneurs is capital. According to Hisrich and O'Brien (1981), women involved in non-traditional businesses such as construction, consultancy, and computers find greater difficulty in obtaining bank loans than those in traditional businesses. Bankers are less interested in granting them loans because they are uncertain with the amount of savings these women entrepreneurs have. Women are also viewed as being less serious in business than men.

Buechler (1995) further argued that women micro-entrepreneurs often feel uncomfortable when dealing with formal financial institutions, which tend to be male-dominated. According to Maysami and Goby (1999), the most common problems facing women entrepreneurs are the lack of start up capital, lack of confidence, and family issues. Teo (1996) found that female business owners in Singapore have difficulty in finding labour, obtaining finance, competing with others, establishing credibility, and coping with high business cost. Van Der Merve (2003) found that among the barriers facing women involved in this industry are lack of access to financial resources, lack of support, negative prevailing socio-cultural attitudes, gender discrimination, personal difficulties, and lack of basic attitudes and skills, such as self-confidence, self-motivation, and communication.

Based on the findings by Verwey (2007), new challenges, need for achievement and success, love for construction and building, and preference for independence are among the top factors influencing women's involvement in construction. Other factors such as motivation by family members, dissatisfaction with previous job, unemployment, and inability to obtain salaried positions are given low ranking by the respondents.

RESEARCH METHODOLOGY

Questionnaire design

Based on the literature review, the questionnaire survey was designed to have three parts. Part A focuses on the company profile, part B on the characteristics of women construction entrepreneurs, and part C on the challenges and problems faced by the respondent. Part B assesses the important characteristics such as number of family members, marital status, formal educational level, involvement in the industry, courses attended, industry opportunities, motivating factor, entrepreneurial skill, and business strategy adapted. Part C evaluates women entrepreneurs in terms of the challenges and problems they face, such as initial capital, gender discrimination, community perception, and challenges and problems they faced in the industry.

Sampling design

Non-probability sampling design, that is, quota sampling was used to select the respondents. The population consists of women contractors registered with the Construction Industry Development Board (CIDB) in different categories, that is, grades one to seven. This study only included 50 women contractor in the Northern part of Peninsular Malaysia, which comprises the states of Penang, Kedah, Perlis, and Perak. A total of 14 responses were obtained, giving a total response rate of 28%. The survey was conducted in January 2006.

ANALYSIS

Table 1 shows the profiles of companies owned by women construction entrepreneurs. Based on the years of operation, most of the companies existed for more than five years in the industry, while 8 or 57.1% existed for more than 10 years. All of the respondents worked full time in their companies. Private limited companies numbered 10 or 71.4% of the companies, while only four companies were sole proprietorship. Partnership was not a popular strategy among the women contractors.

In terms of number of employees, 11 or 78.6% had more than six employees, and only three contractors or 21.4% had less than five employees. Eight or 57.1% of the companies started with an initial capital of less than RM100,000 (RM3.70 is approximately equivalent to US$1.00). Currently, most of the companies (7 or 50%) are involved in medium-scale projects, with 8 or 57.1% of the companies obtaining their project through the open tender.

Table 1. Profile of the companies

Description:	Frequency:	Percentage:
Years of operation		
< 5 years	3	21.4
6-10 years	3	21.4
> 10 years	8	57.1
Type of involvement		
Full time	14	100.0
Part time	0	0.0
Type of business		
Sole proprietorship	4	28.6
Partnership	0	0.0
Private limited	10	71.4
Public	0	0.0
Number of workers		
1-5 peoples	3	21.4
6-10 peoples	4	28.6
11-25 peoples	3	21.4
26-50 peoples	1	7.1
> 50 peoples	3	21.4
Initial capital		
< RM 100,000	8	57.1
RM 100,000-RM 500,000	5	35.7
> RM 500, 000	1	7.1
Number of projects awarded in year 2005		
Micro (<RM200,000)	0	0
Small (RM500,000 to 1million)	4	28.6
Medium (I million to 5 million)	7	50.0
Big (> 5 million)	2	14.3
No project	1	7.1
Type of procurement method		
Selected tender	1	7.1
Open tender	8	57.1
Negotiate tender	4	28.6
Proposal based	1	7.1

Table 2 shows the characteristics of women construction entrepreneurs. Five or 35.70% of the women entrepreneurs were the youngest daughters, with 13 or 92.9% of them married. All possessed various educational levels, with secondary education level being the lowest. Seven or 50.0% were the company founders, while 3 or 21.4% started their businesses based on their previous experience.

The table also indicates that women entrepreneurs are willing to attend seminars or courses in preparation to starting their own businesses. Almost all of the respondents attended business-related courses. Ten or 71.4% agreed that capital is a motivating factor for venturing into the construction business. All received advice from government agencies.

Half of them agreed that the industry provides more opportunities for them, while the other half only mildly agreed with the suggestion. Nine or 64.3% of the respondents agreed that hard work is the most important motivating factor compared to capital, particularly when planning and finding business opportunities. With regard to entrepreneur skill, managerial skill and good networking were chosen as the two most important skills. Three or 21.4% of the women agreed that the inclination to take risk is a must in any business. All the women entrepreneurs had their own business strategies. Seven or 50% wished to diversify their contracting business by opening new branches; 6 or 42.9% hoped to diversify into new types of business; and 1 entrepreneur wanted to implement a backward strategy of becoming a supplier.

Table 2. The characteristics of women construction entrepreneurs

Description:	Frequency:	Percentage:
Number in the family		
First	1	7.1
Second	1	7.1
Third	2	14.3
Youngest	5	35.7
Others	3	21.4
Marital status		
Single	1	7.1
Married	13	92.9
Formal educational level		
Secondary school	8	57.1
Certificate	1	7.1
Diploma	2	14.3
Degree	3	21.4
How did they get involved in the industry?		
Inherited business	1	7.1
Having previous experience	3	21.4
Buy-out of existing business	1	7.1
Partnership	2	14.3
Founder	7	50.0
Important step towards becoming an entrepreneur		
Attended courses	10	71.4
Received enough funding	2	14.3
Registered with specific body	1	7.1
Obtained experience	1	7.1
Attended courses/entrepreneurship training		
Yes	13	92.9
No	1	7.1
Capital as motivating factor?		
Yes	10	71.4
No	4	28.6
Advise from government agency		
Yes	14	100.0
No	0	0.0
Opportunities for women entrepreneur in the industry		
More opportunities	7	50.0
Less opportunities	7	50.0
Motivating factor		
Capital	5	35.7
Hard work	9	64.3
Entrepreneurial skills		

Managerial skill	5	35.7
Good networking	5	35.7
Risk taking	3	21.4
Creativity and innovation	1	7.1
Consistency	1	7.1
Business strategy		
Business diversification through opening of new branch	7	50.0
Diversifying into a new industry	6	42.9
Backward strategy (i.e., becoming a supplier)	1	7.1

Table 3 shows the challenges and problems faced by these women entrepreneurs. The majority of the respondents (7 or 50%) funded the start up of their businesses using their own savings, 4 or 28.60% using bank loans, 2 or 14.30% using their husbands' savings, and only 1 or 7.1% using government agency loans.

Only five or 35.7% of the respondents mentioned discrimination problems in the industry. Six or 42.90% of them had the perception that the community disagrees with women's involvement in the industry, while five or 35.70% of the respondents think that the community has a good perception on their involvement. As this is a male-dominated industry, the majority of the respondents (six or 42.9%) had problems in dealing with men's egoism, while four or 28.6% of the respondents had problems in convincing people in the industry.

Table 3. Challenges and problems in being women entrepreneurs

Description:	Frequency:	Percentage:
Initial capital source		
Own funding	7	50.0
Borrowed from family	0	0.0
Borrowed from husband	2	14.3
Borrowed from friends	0	0.0
Borrowed from banks	4	28.6
Loan from government agencies	1	7.1
Gender discrimination		
Yes	5	35.7
No	9	64.3
Community perception		
Less experience	3	21.4
Not suitable to women	6	42.9
As an employee not employer	0	0.0
Women can compete	5	35.7
Challenges		
Men's egoism	6	42.9
Difficulty in convincing people	4	28.6
Stereotypical perception of clients on the conventional roles of women	2	14.3
Difficulty in obtaining information	1	7.1
Negative community perception	1	7.1

DISCUSSION

This paper had provided a general documentary on women construction entrepreneurs in Malaysia. Descriptive analysis helps to provide data on their involvement in the industry. Even though this study is only based on a small sample size and number of responses, the finding could be useful and relevant for other researchers interested to make in-depth study on women construction entrepreneurs.

This research shows that women entrepreneurs began to enter the industry in the mid-1990s. The economic boom from 1994 to 1996 created many opportunities in the industry, providing the main push factor for the early involvement of the surveyed women entrepreneurs. All were involved as a full-time entrepreneurs in the industry. The majority were small- and medium-scale operators with less than 25 employees and who bid through the open tender. Most of them started their businesses with an initial capital of less than RM100,000.00 and were concentrating on medium-sized projects. These women fit the definition of successful woman entrepreneurs given by Van Der Merve (2003). The findings also support Gale's (1994) hypothesis that women are more concerned with maintaining the status quo as they have to struggle to fit in, prove themselves, and 'win' their place in the male-dominated sector.

In relation to their background profiles, the women entrepreneurs already acquired certain knowledge and a certain level of education before they entered the industry. This study found that 57.1% of the respondents had secondary level of education, and they learned more from attending courses and received assistance from the government. These helped the women learn and understand the construction culture and knowledge of the industry. Unlike other professions, construction deals more with technical works, which entrepreneurs learn through experience. It is likely that their success is supported by industry knowledge. Thus, women entrepreneurs would like to 'fit in' to the industry's culture.

The majority of the women contractors were founders. They initiated their own company based on their own interest, experience, and sources of capital. Looking at their characteristics, the women entrepreneurs were very proactive and well prepared for any challenge. They attended entrepreneurship courses and sought advice from government agencies.

Hard work and capital were their main strengths in starting and running their businesses. The respondents agreed that initial capital is important in starting a business. Half of the entrepreneurs agreed that there are many opportunities in the industry. Hard work can be closely related to the effort in finding opportunities in the industry. This study supports Maysami and Goby's (1999) results that find women business owners in Singapore to be self-disciplined, persevering, and hard working. Further, this study supports Ljunggren and Kolvereid (1996) who found that women entrepreneurs perceive themselves to have a higher internal drive than their male counterparts. Therefore, the results of Verwey (2007), which has been discussed previosuly, can be used to explain the hard working characteristic possessed by women construction entrepreneurs. Another important issue is that women entrepreneurs strongly emphasise their business aspirations. They can foresee their potential business growth because they are strongly focused on business planning.

In terms of challenges, the most important one they have to face is overcoming the problems of insufficient capital. Half of them had to borrow from others to start their businesses, while the other half used their own funds. This result is in accordance with that of previous research, which suggests that female entrepreneurs use less capital, consequently tending to start small enterprises (Carter and Rosa, 1998) and to secure less external finance (Coleman and Carsky, 1996). Some of the women entrepreneurs heavily depended on bank loans. This study also supports Jaafar (2003) and Van De Merve (2003). They claimed that contractors heavily depend on external debt, which normally refers to bank loans. The construction industry has a low entry barrier, particularly in terms of capital. As it does not require large capital (Jaafar, 2003), it provides large opportunities for entrepreneurs to start a company. Furthermore, the increment of sub-contracting practice and the 'pay when you have been paid' system require entrepreneurs to be good in management than to have a large amount of capital and a specific technical expertise. This helps explain why women entrepreneurs can succeed even if they only possess secondary educational level.

Another challenge that women have to face is convincing other parties in the industry of their leadership capability as women do not traditionally lead construction companies. Men may not accept the fact that women can manage projects as good as them. Community perception is unclear with regard to the involvement of women entrepreneurs. Even though women's involvement is negligible, there is no gender discrimination in terms of the assistance provided by government agencies in Malaysia.

According to Gale and Susan (1995), education acts as a 'gatekeeper' that can usher women into the construction culture. Nowadays, almost 70% of public university students are female. The high proportion of females among students in construction-related courses in universities, private institutions, and polytechnics can be considered an important contribution to women's involvement in the industry. Furthermore, in Malaysia, there is an inreasing number of universities and colleges offering courses related to construction. All these indicate that in the future, there will be more women in the workforce as well as women entrepreneurs in the industry.

CONCLUSIONS AND RECOMMENDATION

The present findings suggest that the construction industry should provide empowerment for women to enhance their involvement in business. It has been forecast that Malaysia's population will be 34 million by 2020, wherein 60% will be urban based. This will increase the demand for houses, transportation, education, water, and other infrastructure facilities very closely connected to the construction industry. Thus, the construction industry is expected to provide good opportunities for entrepreneurs.

Because of the small sample size, the findings should be treated with circumspection. The result are not robust. Nonetheless, the findings can be of use for future research undertakings on the subject matter. The reason for their successful participation can be attributed to their having formed network relationships in the industry early on. Despite the presence of many current 'barriers' for their involvement, women entrepreneurs' close network and relationship with industry players can stimulate their motivation to grow. Business organisation such as chambers of commerce and other related associations should play an active role in attracting women involvement in the industry. Women's active involvement in business can help enhance the industry's economic growth.

Based on the above discussions, the research indicates that women construction entrepreneurs are ready, more independent, hard working, can plan well, and can achieve success in a male-dominated industry. As this study only covered a small population of women contractors in the Malaysian construction industry, the findings cannot be generalised to all women contractors in different contexts. A thorough research needs to be conducted to obtain a clearer picture of women's involvement in the construction industry.

REFERENCES

Abdullah, M. A. (1999) *Pembangunan Perindustrian di Malaysia.* Kuala Lumpur: Fajar Bakti.

Ahl, H. (2006) Why Research on Women Entrepreneurs Needs New Directions. *Entrepreneurship, Theory and Practice*, Sept, 595-621

Bracker, J., Keats, B. and Pearson, J. (1988) Planning and Financial Performance Among Small Firms In A Growth Industry. *Strategic Management Journal*, 9(6), 591-603

Buechler, S. (1995) The Key to Lending to Women Microentrepreneurs. *Small Enterprise Development*, 6, 213-225

Brush, C. (1990) Women and Enterprise Creation: An Overview of Women-Owned Business. In Enterprising Women: Local Initiatives for Job Creation. Paris: Organisation for Economics Co-operation and Development (OECD), 37-55

Brush, C. G. and Hisrich, R. D. (1991) Antecedent Influences on Women-Owned Businesses. *Journal of Managerial Psychology*, 6 (2), 9-16.

Buttner, E. H. (2001) Examining Female Entrepreneurs' Management Style: An Application of a Relational Frame. *Journal of Business Ethics*, 29 (3), 253-269

Carter, S. and Rosa, P. (1998) The Financing of Male and Female Owned Businesses. *Entrepreneurship and Regional Development*, 10(3), 203-24

Coleman, S. and Carsky, M. (1996) Financing Small Business: Strategies Employed by Women Entrepreneurs. *The Journal of Applied Management and Entrepreneurship*, 3 (1), 28-42.

Cunningham, J.B. and Lishesrun, J. (1991) Defining Entrepreneurship. *Journal of Small Business Management,* 31 (1), 45-60.

Deng, S., Hassan, L. and Jivan, S. (1995) Female Entrepreneurs Doing Business in Asia: A Special Investigation. *Journal of Small Business and Enterprise,* 12, 60-80.

Dolinski, A. L., Caputo, R. K., Pasumaty, K. and Quanzi, H. (1993) The Effects of Education on Business Ownership: A Longitudinal Study of Women Entrepreneurship. *Entrepreneurship Theory and Practice*, Fall, 43-53.

Equal Opportunities Commission (EOC). (1990) Women and Men in Britain, HMSO, London

Feather, N.T. (1988) From Values to Action: Recent Application of the Expectancy-value Model. *Australian Journal of Psychology,* 40(2), 377-390.

Gale, A.W. (1991) The Image of the Construction Industry: How Can It Be Changed?. *Proceedings of the Seventh Annual ARCOM Conference*, University of Bath, September, 191-202.

Gale, A. W. (1992) The Construction Industry's Male Culture Must Feminize if Conflict is to be Reduced: The Role of Education as a Gatekeeper to a Male Construction Industry. In Ed. Fenn, P and Ganeson, R. *Construction Conflict: Management and Resolution*: London, Spon, 416-427.

Gale, A. W. (1994) Women in Non-Traditional Occupations: The Construction Industry. *Women in Management Review*, 9 (2), 3-14

Gale, A. W. and Susan, C. (1995) Women in Project Management: Entry Into a Male Domain? A Dis. *Leadership and Organization Development Journal*, 6 (2), 3-9.

Hisrich, R. D. and O'Brien, M. (1981) Women entrepreneurs in a developing economy, Wallesley; Bobson College, Mass.

Holmquist, C. and Sudin, E. (1984) Women as Entrepreneurs in Sweden: Conclusion From A survey, Bobson College, Mass; Wallesley.

Jaafar, M. (2003) *Critical Success Factors for Small and Medium Sized Contracting Firms.* Ph.D. Thesis, Universiti Sains Malaysia.

Kamaruddin, Z. (1997) *Wanita Dalam Perlembagaan Malaysia,* Kuala Lumpur:, ANZ Charisma.

Karim, S. (1997) *Wanita, Profesion dan Keluarga*. Kuala Lumpur: Pelanduk Publication.

Kartz, V. (1989) *Characteristics and Background of Entrepreneur*. Boston; BPI Irwin

Kotey, B. and Meredith, G. G. (1997) Personal Values, Business Strategies and Enterprise Performance. *Journal of Small Business Management,* 35(2), 37-43.

Ljunggren, E. and Kolvereid, L. (1996) New business formation: Does gender make a difference?. *Women in Management Review*, vol. 11, no 4, 3-12

Mamat, Kamaruddin and Ramli. (1990) The Malaysian Entrepreneur- His Qualities, Attitudes and Skills. *Malaysian Management Review,* 25, 11-18.

Maimunah, l. (1996) Gender Needs Analysis of Women Entrepreneurs. *Pertanika Journal of Social Science and Humanities*, 4(1), 1-9

Maysamy, R. C. and Goby, V. P. (1999) Female Business Owners in Singapore and Elsewhere: A Review of Studies. *Journal of Small Business Management*, 37 (2), 96-107

Nordin, M. (2005) Woman in Business: Determinants for venturing in Malaysian SMEs. Retrieved Dec 6, 2009 from www.tbs.ubbcluj.ro/studia/articol_4_2_2005.pdf

Shim, S., and Eastlick, M. A. (1998) Characteristics of Hispanic Female Business Owners: An Exploratory Study. *Journal of Small Business Management*, 36 (3), 18-29

Smeltzer, L. R. and Fann, G. L. (1989) Gender Differences in Exteral Networks of Small Business Owner/Managers, *Journal of Small Business Management,* 27 (2), 25-32.

Statistical Department Malaysia, 1999.

Teo, S. K. (1996) *Women Entrepreneurs of Singapore, Singapore Business Development Series. Entrepreneurs, Entrepreneurship and Enterprise Culture,* Ed. Low and Tan. Singapore. Addison-Wesley Publishing Company, 254-289.

Teoh, W. M. Y. and Chong, S. C. (2007) Women entrepreneurship in Malaysia: A Framework of factors influencing performance. *The 7th Asian Academy of Management Conference,* 22-24th May 2007, Penang, Malaysia.

Van Der Merve, M. (2003) *Women Entrepreneurs in South Africa*. Unpublished MPhil Entrepreneurship course material at the faculty of economic and Management Sciences, Department of Business Management, University of Pretoria.

Vervey, I. (2007) Success Factors of Women Entrepreneurs in Construction. *Paper presented in CIB World Building Congress 2007*. Capetown, South Africa.

Vester, N. (1980) *New Venture Strategies*; Englewood Cliffs, New Jersey.

Watkin, J. M. and Watkin, D. S. (1982) The Female Entrepreneur: Her Background And Determinants of Business Choices: Some British Data", Mass, Wellesley: Babson College. Watson, J. (2003). "Failure rates for female-controlled businesses: Are they any different?. *Journal of small business management*, 41 (3), 262

Watson, J. (2002) Comparing the performance of male and female controlled businesses: relating outputs to inputs. *Entrepreneurship, theory and Practice*, 26 (3), 91-100.

AN ANALYSIS OF PAYMENT EFFECTED BY STANDARD FORMS OF CONSTRUCTION CONTRACT IN SARAWAK, MALAYSIA: A COMPARATIVE STUDY OF PWD 75, CIDB 2000 AND PAM 1998

Ting Sim Nee[1], Sia Pik Soon[1], Andrew Whyte[2]
[1]*Department of Civil Engineering, Universiti Malaysia Sarawak, 94300 Kota Samarahan Sarawak, Malaysia,*
[2]*Department of Civil Engineering, Curtin University of Technology, Perth WA, Australia*

Abstract
In Malaysia, there are several standard forms of general conditions of contract used as formal tools for contractual relationships between the parties in the local construction industry. The purpose of this study is to create a better understanding of the explicit clauses in alternative Standard Forms relating to payment in general and how, in their application, construction contract administration effects efficient project realisation. Contract clauses are examined towards a body of work that seeks to update and review and identify contract improvement suggestions. Analysis was made of three principal standard forms in East Malaysia, firstly the most commonly used and dominant Standard Form in Sarawak, that of the Malaysian Public Works Department 75 (PWD 75), secondly the CIDB Standard Form of Contract for Building Works 2000 Edition (CIDB 2000) and, thirdly the Malaysian Standard Form of Building Contract 1998 Edition (PAM 1998 Form). Theoretical comparisons were made and semi-structured interviews were carried-out to investigate practice and application in industry with regard to *completeness and comprehensiveness*. Findings show that the prevalent standard form in Malaysia's largest state fails to clarify issues such as penultimate claims, account preparation procedures, time frames for settlement and submission of final claim, leading to a greater propensity for dispute and conflict in the realisation of the client's brief. Findings offer a contribution to this field of construction contract administration in Malaysia. It also set the stage for further and necessary studies into construction contracts in Malaysia, where majorities of construction projects are carried out with standard forms. Problems as such shown in findings are prevalent in the industry and this indicates that improvements and changes are crucial in order to bring betterment to the industry and all the contracting parties.

Keywords: C*onstruction-contracts; standard-forms; payment; Sarawak-Malaysia*

INTRODUCTION

In whichever capacity a practising civil engineer is working within the construction industry, irrespective of the type of organisation, it is likely that they will be operating in a contractual environment (Williams, 1992). An engineering construction contract refers to an Agreement, which is enforceable by law, executed between the Employer and the Contractor together with the documents forming the proposed scope of construction work. In Malaysia, such contract is a promise or a set of promises that are enforced under the Contract Act 1950 (Act 136).

It is a fundamental rule of construction that a contract must be interpreted as a whole, with the meaning taken from the entire context rather than from a consideration of only particular portions or clauses of the agreement (Bockrath, 2000). The purpose of a contract is to govern the rights, duties and liabilities of the builder who performs the work, the person or organisation for whom the construction is to be executed, and the architects and engineers who design and perform contract administration and inspection (Bockrath, 2000). In addition, the contract also defines the scope of the work and the Employer's delegation of responsibilities to the various parties to complete the project, the nature and extent of risk to the various parties involved, and the financial incentives to complete the construction work.

Standard forms

One of the important documents in forming an engineering construction contract is the Conditions of that Contract. The Conditions of Contract set out the legal and contractual constraints applying to the agreement to build and these represent the main operative provisions in the contract (Rajoo, 1999). According to Ashworth (1998), the Conditions of Contract seek to establish the legal framework under which the construction work is to be undertaken.

In essence, the Conditions of Contract are the 'rules' governing the execution of the contract. Specifically, they define the powers and rights, responsibilities and obligations of the parties involved in the contract. Additionally, they detail the rights of the stakeholders to the contract in that they specify the various courses of action which are open to either party in the event of the other party's failure to discharge their contractual obligations (Williams, 1992). In an engineering construction contract, the constituent clauses of Conditions of Contract should be written with great care so as to be clear, precise and unambiguous, since the Conditions of Contact constitute much of the legal basis on which any court ruling would be made in the event of a dispute (Williams, 1992).

The general conditions of a construction contract are, as the name implies, those terms and conditions of a general nature that apply to the work as a whole (Collier, 2001). They are generally suitable for a wide range of common projects or works. These general conditions are deemed to be agreed and are not subjected to further negotiation and amendment (Singh, 2002), the insertion of so-called addendum special-clauses notwithstanding.

The conditions of contract will generally be arranged in a printed form, commonly referred to as the standard form of contract and, published by an authoritative body of the industry recognised by all parties. The main purpose of using standard forms of general conditions is to provide a basic legal framework for the administrative procedures necessary to affect the legal and commercial relationship between the parties for achieving the purposes of the contract (Singh, 2002).

Standard forms in Malaysia: payment clauses

In Malaysia the usage of standard forms of contract in construction is extensive. These fundamental standard forms are largely categorised according to their specific main purpose. For example: the Public Works Department (PWD) Forms are used for works in the public sector; the Pertubuhan Akitek Malaysia (PAM) Forms are essentially used for private sector building projects; and, the Construction Industry Development Board (CIDB) Forms, are intended for use in both public and private sector projects.

Public Works Department 75 (PWD 75) is the standard form of general conditions of civil engineering contract used extensively in the public sector construction projects in Sarawak, Malaysia's largest state. The PWD 75 under study was revised in May 1961. It was originally based and closely modelled on the Royal Institute of British Architects (RIBA) Standard Form of Building Contract, which was a common standard form of conditions of contract used in United Kingdom. It is important to note that the Public Works Department 75 (PWD 75) standard form of contract is, as it has remained for the past 40 years, the principal public sector standard form in use in the state of Sarawak.

Due to the independency of the politic and as agreed in constitution when the state of Sarawak joined the Malayan Union to form Malaysia in 1963, the state government of Sarawak is independent and has full authority in certain fields, with public works being one of them. Hence, PWD 75 is still used extensively in all the public sector projects in Sarawak, including all federally-initiated projects, state-initiated projects and other projects in related to the state government. PWD 75 is also widely applied in private sector projects in Sarawak. The private companies, clients or semi-government authorities, who use the PWD 75 as the conditions of contracts, would maintain most of the clauses while making minor amendments to suit their projects in a somewhat piecemeal invalidated fashion.

The Public Works Department (PWD) or as nowadays preferred as Jabatan Kerja Raya (JKR) has taken the initiative to revise on this contract document since year 2000 and rolled out a PWD 75 (Version 2006) in the year 2007. Despite a 2006 revision[1], it has yet to be fully rolled-out in Sarawak, nor embraced by a state industry conditioned by four decades of PWD 75 dominance. With information gathered from Sarawak's Public Work Department, many private and governmental organisation are currently still using the older version due to reasons such as unfinished projects, familiarity with old forms and in some cases, no awareness of the existence of the new form despite effort to promote its usage.

In a typical engineering and construction standard form of contract, the effecting of payment to the contractor in return for performance is one of the primary obligations of the employer and the right of the contractor. One of the most common methods of effecting payment to the contractor is through the process known as 'interim' periodic payment, to relieve the contractor of the burden of a totally negative cash-flow, the cost of which may otherwise be an overhead reflected in the tender sum.

Interim payments are effected by the issue of 'interim certificates': an approximate indication of the value of work executed and the corresponding amount due to the Contractor. If there is any over-certification or under-certification, there can be a commensurate revision in the subsequent certificate. Failure of the contract administrator to issue the relevant interim certificates, in line with the stipulations of the contract, can expose the employer to a possible claim of breach of contract by the contractor.

Owing to the significance of payment and interim certificates, all alternative standard forms of general conditions of contract in Malaysia have incorporated payment provisions. These are tabulated in Table 1.

[1] It is worth noting that a separate structured analysis, by the authors, of the implications and effects of the updated and revised 2006 version of PWD 75 is available under separate cover and compliments the work presented here.

Table 1. Malaysian standard forms of contract: payment clauses

	Malaysian Standard Forms of Contract		
	PWD 75	**CIDB 2000**	**PAM 1998**
Principal clauses related to Payments	**35**	**42, 43**	**30.0**

The inherent advantages in the use of standard forms contribute to their widespread use in industry: they eradicate the cost of preparing project-specific contracts; assist tendering; balance risk; and, clarify roles. However, whilst standard forms do reduce conflict, they do not eliminate it completely, particularly where payment is concerned. The main standard forms in Malaysia experience discrepancies in the interpretation of the clauses of payment. Issues related to interim certificates and payments have always been problematic for contract administrators; and according to Danuri et al. (2008) claim and dispute over payment are the common sources of disputes. And it may be worth noting that in relation to payment disputes alone, findings from questionnaire survey show that between 2000 and 2005 more than 60% of Malaysian Contractors have experienced late payment problems, whether in government-funded projects or private projects (Danuri et al, 2008).

Given the occurrence of discrepancy between parties to a particular project, the most readily available standard forms in Sarawak, Malaysia of PWD 75, PAM 1998 and CIDB 2000, require further study; particularly those clauses relating to payment and how they effect efficient, appropriate and timely remuneration for construction of the works. The study presented in proceeding section provides analyses towards this problem, and examines the issues related to payment through an analysis of the relevant clauses of the Malaysian standard forms of contract and their application in the local industry.

METHODOLOGY

An extensive theoretical analysis of the three principal standard forms of contract in Malaysia was carried out as a necessary step towards a review of the antecedents of conflict related to payment. Given its continuing dominance of the industry in Malaysia's largest state, the original PWD 75 formed the core of the analysis which sought to establish:

i) The identification and review of the relevant payment clauses
ii) The rights of the parties governed by those relevant clauses
iii) The obligations of the parties stated in the relevant payment clauses

The analysis of PWD 75 was then repeated for PAM 1998 and CIDB 2000 forms.

Comparisons were then made among the three (3) forms with respect to firstly, completeness (whether the clauses cover all the basic and necessary matters under payment) and secondly, comprehensiveness (whether the clauses could be used in most payment related problem scenarios and situations) of the clauses related to payment.

Following the theoretical analysis and comparison of explicit clauses related to payment, a sample of industry stakeholders were interviewed using a semi-structured interviewing format to draw out application of, and issues of contention in, the payment clauses as they relate to construction contract administration and practices in Malaysia.

Sample respondents were identified, ranked (by experience, qualification and local industry participation) and then selected to represent the various parties involved in project realisation, namely: employer/owner/client; consultant/superintendent/engineer/architect; and, contractor/builder/subcontractor.

Those identified as suitable for participation in subsequent structured-interviews had at least 15 years experience in civil engineering construction works and contract administration in particular. They each held senior positions in their respective organisations and had a solid working-knowledge of each of the three (3) standard forms of contract under study.

The work presented here followed a qualitative research methodology (Vogt, 2007). In other words the systematic collection of data involved an interpretive, naturalistic approach to the subject matter and concentrated on the examination of a variety of empirical studies based upon personal experience, introspection, and historical, interactional and visual text. Whilst this qualitative approach was chosen purposely to gain from information generated by an extensive open-ended research process, it is recognised that this might result in potential limitations of consistency and objectivity (Creswell, 2005). Limitations, whilst acknowledged, are argued not to detract from the results described below.

RESULTS AND DISCUSSION

The following sections identify and review explicit clauses of certification and payment in each of the three standard forms of contract under study, and detail application issues experienced in the field. This academic examination was carried out as part of the work of an overall examination of all the contract clauses in the stated standard forms. Contract clauses are examined towards a body of work that seeks review and contract improvement suggestions to reduce conflict in the construction industry in Malaysia in general and Sarawak in particular. It is envisaged that such analysis could be strengthened by further examinations of other local and international standard forms with the ultimate aim of establishing a complete set of Malaysian form that can be used by all construction contracting parties in Malaysia, including foreign-initiated/funded projects.

PWD 75

Public Works Department 75 is the dominant standard form of contract used locally. Clause 35 in PWD 75 deals with all interim certificate matters and regulates payment under this form. It governs all aspects of the subject of payment including interim certificates, retention monies, final account and the effect of certificates.

Clauses 35(a) and 35(b) cover the provisions in respect of interim certification. Clause 35(a) provides for the issue of interim certificates by the Engineer. The Contractor is entitled to the issue of such certificate at the Period of Interim Certificates as prescribed in the contract, and is then entitled to payment of the amount so stated in such certificate within fourteen (14) days of the issue of the certificate. However, such certificate is only issued within fourteen (14) days after the Contractor submits a detailed written application to the Engineer.

Semi-structured interviews reveal however that, in common practice, payment to the Contractor is seldom made by the Employer within fourteen (14) days; the time period stated in this clause is seldom adhered to locally. Another 'more reasonable' timeframe is argued as needed to update PWD 75; thirty (30) days for payment pursuant to the issue of interim certificate, is suggested by the Public-Sector Employer's Respondents, to make payment to the Contractor, given that this represents current practice in industry.

Regarding the submitted written application for interim payment, the Engineers-Group Respondents are of the opinion that the Contractor does not necessarily need to submit a detailed written application, since the Superintending Engineer is best placed to assess work done; however, it is acknowledged that the submission of a detailed written application by the Contractor does make the Engineer's life easier, albeit allowing the potential for a passively perilous resting-on-laurels, particularly if the Engineer is not required to issue any such certificate if the amount due does not achieve a minimum amount as stated in the contract.

The Study Respondents had no further issue with Clause 35(b) which deals with valuation of interim certificates, where the amount so certified includes the total value of the work properly executed and materials or goods delivered to the site for use in the Works, up to and including a date not more than seven (7) days before the date of the certificate, given that the payment must deduct retention and previous instalments.

Clauses 35(c) and (d) are related to retention monies to be help by the Employer. Clause 35(c) authorises the Employer to retain 10% of the total value of the work and materials in every Interim Certificate, and up to the amount provided in the contract as the Limit of Retention Fund. When the amount so retained reaches such limit, the Engineer must certify the full value of the Works and materials in payment certificates. The analysis of this clause found the Clients' Respondents suggesting that, in practice, the Employer does not necessarily inform the Contractor in writing of a reason, if he exercises any right of deduction from monies due under the contract, as the Contractor should know the reasons behind such deduction and what amount is to be deducted. It might be argued however that for the sake of transparency, clarification in the clause, of cause, might assist acceptance of deduction.

The respondents had no further issue with Clause 35(d) which deals with the release of the retention fund, where the Engineer issues a certificate for release: for example half of the retention fund, on the practical completion of the Works or of any section thereof, and the other moiety (which might be replaced by the less archaic term of portion) to be included in the final certificate. Although again, the somewhat less than reasonable time-frame of Contractor's entitlement to payment within fourteen (14) days of the issue of the certificate, might be an option for review argued the Study Respondents.

Clause 35(e) covers the matters concerning the settlement of accounts, whilst Clauses 35(f) and (g) contain provisions with regard to final certificate, respectively: issuance; and, nature and effect. Under Clause 35(f), the Contractor should be given an opportunity to check the valuation stated in the final certificate before its issue. Although there is no specified time frame for the Employer to make final payment to the Contractor, the Stduy Respondents comment that normal practice is that the Employer makes final payment to the

Contractor within twelve (12) months after the Defects Liability Period. Again, it is argued that to avoid dispute, explicit expression might enhance administration under PWD 75.

Clause 35(g) deals with the nature and effect of the final certificate. Subject to three specified exceptions and unless notice of arbitration has been given before its issue, this clause states that this final certificate should be conclusive evidence in any proceedings arising out of the contract. The sections below (3.2 and 3.3) expand upon the Study Respondent's opinion that this may distort the balance of risk in favour of the Contractor. Clause 35(h) provides that apart from the conclusive nature as ascribed to the final certificate by Clause 35(g), no other certificate is considered as conclusive evidence that any work, materials or goods are in accordance with the contract.

CIDB 2000

Like the other standard forms of conditions of contract, CIDB 2000 contains contractual provisions dealing with certificates and payments. Clause 42 stipulates procedures for payment claims to be made and assessed.

Clause 42.1 requires the Contractor to provide the Superintending Officer (SO) with a Statement of Work Done for amounts up to the last day of relevant interval. In contrast, Clause 35(b) of the PWD 75 provides that the amount so stated in the Contractor's application is up to a date not more than seven (7) days before the date of such application. The Contractors' Respondents interviewed in this study, argue that the CIDB clause reflects better a more timely payment of work done.

The Respondents also argue a general point of caution that, in practice, the determination, definition and qualification of an SO, and consequently their power for certification, may be more open to debate than the CIDB authors envisaged.

Clause 42.2 is related to the issuance of interim certificates. This sub-clause authorises the SO to issue an Interim Certificate within twenty-one (21) days of receiving the Statement of Work Done, as compared to within fourteen (14) days in Clause 35(a) of the PWD 75, arguably a more practicable time-frame. The amount stated in an Interim Certificate would be the total amount stated in Statement of Work Done, less retention monies, any amount previously certified and any sum certified to be deducted under the contract. In contrast, Clause 35(b) of the PWD 75 does not include the last item, which is identified as an area of contention by the Study Respondents.

Sub-Clause 42.2(c) provides that the Interim Certificate must be issued at regular intervals as stated in the contract. In comparison with PWD 75, a notable addition to this sub-clause is that the SO may issue interim certificates as and when he has ascertained that further amounts are due to the Contractor after the practical completion of the Works. Sub-Clause 42.2(e) is a provision that is not found in PWD 75, providing that the SO must notify the Contractor in writing if he is not issuing an Interim Certificate. It might be argued here that these stipulations go towards transparency of action.

Clause 42.3 deals with the treatment of retention monies. The provisions discussed in Sub-Clauses 42.3(a) and (b) are broadly similar to Clause 35(c) of the PWD 75, except that the Limit of Retention Monies in CIDB 2000 would be 5% of the Contract Sum if none is provided in the contract; PWD 75 does not specify. Another significant feature in the CIDB Form is the requirement under Sub-Clause 42.3(c)(ii) for the Employer to deposit the amount of retention monies deducted in a separate banking account held in trust by the Employer, so as to protect it in the event of the Employer's insolvency, or to deflect a contractor's claim over possession of its interest accrual. Case-law highlights previous discrepancies between contractor and client on this issue and perhaps justifies the inclusion of this CIDB clause (MLJ 444).

CIDB 2000 Sub-Clauses 42.3(c)(iv) and (v) deal with the release of retention monies that go beyond Clause 35(d) of the PWD 75 in terms of the SO's issuance of an Interim Certificate certifying the release of retention monies within fourteen (14) days following the issue of the Certificate of Practical Completion or the Certificate of Making Good Defects, with remaining moiety (portions) as an option either within an Interim Certificate or as an inclusion in the Final Certificate as appropriate.

CIDB Clause 42.4 again enhances PWD 75 Clause 35 with an additional provision to cover the correction of certificates. Clauses 42.5 to 42.8 outline the procedures for payment on the completion of the Works. Clauses 42.5 and 42.6 dealing with Statement at Completion and Penultimate Certificate are enhanced provisions not available in PWD 75 Clause 35; these clauses are generally seen as positive additions by the Study Respondents.

Clause 42.7 sets out in detail the procedures for submission of the Final Claim Statement, and in comparison to Clause 35(f) of the PWD 75, advises that any default of the Contractor may cause the SO to prepare the Final Account.

The provisions contained in Sub-Clause 42.8, preparation of Final Account and the issue of Final Certificate, are quite different from, and in the Respondents view considerably clearer than, those stated in Clause 35(f) of the PWD 75, particularly with respect to the explicitly stated time-frame.

Clause 42.9 dealing with the Period of Honouring Certificate states that the Employer should pay the amount certified in any issued certificates to the Contractor within twenty-one (21) days or such other time period as may be provided in the contract, contrasting with Clause 35(a) of the PWD 75 Clause 35(a) fourteen (14) days of the issue of any certificate. Again to address the potential for discrepancy over claim to the accrual of interest [related to the discussion above 42.3(c)(ii)] CIDB 2000 Sub-Clauses 42.9(b) and (c) extend provisions in PWD 75 by stating that the Contractor is entitled to interest in respect of the unpaid amount in the event that the Employer fails to make payment to the Contractor within the Period of Honouring Certificate, and that the Contractor also has recourse to recover any unpaid amount from a Payment Bond.

CIDB Clause 42 subclauses 10, 11, 12 and 13 go beyond Clause 35 of the PWD 75, to give additional provisions related to, respectively: remedy for default in payment; recovery deductions recorded as payments; delays in certification; and, quality in respect to the Final certificate. The Respondents to this study argue that this seeks to reflect an intention to clarify the interests of the contractor.

The effect of the Superintending Officer's certification is dealt with in Clause 43 and is largely consistent with PWD 75, although Clause 43.1 stipulates explicitly that no certificate of the SO should be considered as conclusive evidence regarding sufficiency of the works. Clause 43.2 advises the effect of the Final Certificate in various situations involving mediation, arbitration or other proceedings pursuant to the particular provision of the contract, somewhat less stringently as a result of the inserted subclause 43.2(c), than that covered in Clause 35(g) of the PWD 75. For instance, although sub-clauses 43.2(a) renders the Final Certificate as conclusive evidence with regard to quality of works and the like in connection with the contract, CIDB Clauses 43.1 and 43.2(b) and (c) do allow an arbitration process to progress without prejudice that the Final Certificate is conclusive.

Overall, the Respondents to this study comment that general comparisons between PWD 75 and CIDB 2000 highlight that the later document attempts to identify, review and clarify a perceived imbalance of risk conceded by Contractors.

PAM 1998

Pertubuhan Akitek Malaysia (PAM) Standard Forms of Contract, used essentially for private sector building projects, detail Clause 30 as covering matters related to certificates and payment under the contract, with Clause 30.1 describing issuance of certificates and the provision for the (superintending) Architect to correct previous intermediate certificated discrepancies.

As with the discussion above, timescales related to interim certification vary somewhat (PAM 1998: monthly; PWD 75: unspecified unless appended), as do certification periods after Contractors' written application (PAM 1998: unspecified; PWD 75: within 14 days), as well as Periods of Honouring Certificates (PAM: by agreement or 14 days; PWD: stated as 14 days). Clause 30.2 contains an additional provision authorising the Architect to make interim valuations whenever considered necessary. On the other hand, this sub-clause in the PAM 1998 Form does not include the provision with respect to a minimum amount for the Interim Certificate to be issued, which is found in corresponding Clause 35(a) of the PWD 75. The Respondents to this study suggest that whilst greater flexibility is implicit theoretically, this sub-clause is somewhat redundant since monthly valuations remain the norm in practice.

Interim certification coverage between the two standard forms are consistent, although a notable difference is that PAM 1998 Clause 30.3 does not dictate direct payment made by the Employer to the Nominated Sub-contractors, as is found in the corresponding PWD 75 Clause 35(b). Another difference is the Contractor must include all value of the work up to seven (7) days prior to the date of the Interim Certificate but in PWD 75, such period is up to seven (7) days before the date of the Contractor's written application, with, as argued by the Study's Respondents, potential implications for an exclusion of work done until the next time.

Two sub-clauses of Clause 30.3 adopt as express terms of the contract the position as decided by the Federal Court,; these are not covered in Clause 35(b) of the PWD 75. Sub-clause (i) stipulates that the Employer can only withhold or deduct any sum from amounts certified in the Interim Certificates provided such withholding or deduction is expressly provided for in the Conditions or certified by the Architect. Payment of the amounts certified in any certificate must be made. Sub-clause (ii) further provides that any disputes or differences as to the certificates should go straight to arbitration. As discussed above, the Study Respondents argue that these sub-clauses go towards addressing a perceived risk imbalance conceded by the Contractor in PWD 75.

Retention monies described in Clauses 30.4 and 30.5 are consistent with PWD 75 (PAM 1998: by appended agreement; PWD 75: 10%) Like CIDB 2000, the Limit of Retention Fund in PAM 1998 Form would not normally exceed 5% of the Contract Sum if none is stated in the Appendix.

Clause 30.5 [(i) and (ii)] set out the nature and purpose of retention and the rules on its treatment and, in essence, duplicate CIDB 2000 [Sub-Clauses 42.3(c) (i) and (iii)]. It is argued here that the stated need to justify and make transparent the Employer's right to deduct described by PAM and CIDB, is found to be lacking in PWD 75 Clause 35.

The release of retention is somewhat similarly stated in the PWD75 Clause 35(d) and PAM 1998 Clauses 30.5(iii) and (iv). Although Sub-Clause 30.5(iv) does require the (superintending) Architect to issue a Certificate for the residue amounts (moiety of retention funding) retained on expiry of the Defects Liability Period or on issuance of the Certificate of Completion of Making Good Defects, whichever is the later. In contrast, Clause 35(d) of the PWD 75 requires the Engineer to include the remaining amounts of the retention fund in the Final Certificate and, by implication, PWD 75 extends the period of Contractor's cash-flow.

PAM 98 Clause 30.6 bears a similarity to Clause 42 of the CIDB 2000 and sets out the procedures and the various steps needed before the Final Certificate can be issued, namely: measurement/valuation of works 6 months from practical completion with Contractor to assist [under 30.6(i)] with supporting documentation; and, issuance of a penultimate certificate of payment with summary copy of valuation to Contractor. In comparison with Clause 35 of the PWD 75, the procedures and time-frames outlined in PAM 1998 are argued, by the study's Respondent Group, to clarify requirements greatly.

Clause 30.7 provides a timetable for the issue of the final certificate and extends PWD 75 Clause 35(f) with a requirement that: the Contractor forwards relevant preparatory documentation; clarifies the issuance of the final certificate within three months of specified events; and, requires that subject to any deduction authorised by the conditions, the balance shown is a debt payable by either party on the 14th day after the issue of the final certificate. As discussed previously, PWD 75 Clause 35(g) identifies the conclusive nature ascribed to the issuance of the final certificate (not only of evidence of the adjustment to the Contract sum, but also that work, materials and goods are in accordance with the contract). PAM 1998 Form does not replicate this condition dealing with the ultimate nature and effect of the final certificate, however it is worth noting that its predecessor, the superseded PAM/ISM 1969 Form did contain a similar condition to PWD 75 Clause 35(g). The Study's Employers' Group of Respondents argue that the updated PAM 1998 form that omits this condition is better, since this has the potential to excuse the Contractor from liability for what could be serious breaches of contract (subject to the limited exceptions). In other words, in line with the risk apportionment philosophy of the PAM 1998 Form, the condition was rightly amended to remove the conclusiveness of the final certificate, and to address the potential for dispute illustrated in case law both locally and overseas (MLJ 16, All ER 121,). Carrying on from the discussion above, Clause 30.8 indeed clarifies that no certificate issued by the Architect is conclusive evidence that any work, materials or goods are in accordance with the contract.

Overall, the Respondents to this study comment that general comparisons between PWD 75 and PAM 1998 highlight that the later document clarifies procedures, whilst maintaining a positive employer's position.

SUMMARY

Based on the above discussion it is generally suggested that stakeholders in the Sarawakian construction industry might benefit from:

- Acknowledgment of the alternative contract forms and project suitability review;
- Explicit standard form clauses that address penultimate claims, account preparation and time frames.
- Inclusions of scenario-based clauses for matters such as payment and interim certification
- Procedural formats with simple language to enhance understanding among participants.

Notwithstanding these general points, Table 2 summarises the specific differences in payment procedure that exist between the three forms of contract under examination in this study.

Table 2. Summary of clauses on certificates and payments

	PWD 75	CIDB 2000	PAM 1998
Relevant clauses	35	42, 43	30.0
Interim Certificate Documentation submitted for issuing certificates	Detailed written application	Statement of Work Done	Details & particulars
Amount due in Interim Certificates	≤ 7 days	Last day	≤ 7 days
Period of Interim Certificates	Self-fill in	1 month if none stated	1 month if none stated
Time frame for issuance after submission of documentation	14 days	21 days	Not specified
Minimum amount stated in Appendix	Yes	Yes	No
Notification if no certificate issued	No	Yes	No
Period of Honouring Certificates	14 days	21 days if none stated	14 days if none stated
Suspension for non payment	No	Yes	No
Retention Monies Retention Percentage	10%	10% if none stated	≤ 10% if none stated

Limit of Retention Fund	Self-fill in	5% of CS if none stated	≤ 5% of CS if none stated
Employer's interest be fiduciary as trustee	No	Yes	Yes
Separate bank account	No	Yes	No
Contractor's beneficial interest	No	Yes	Yes
Contractor to be informed in writing of deduction	No	Yes	Yes
Release of one moiety	Yes	Yes	Yes
Release of second moiety	Final Certificate	Interim Cert. or Final Cert.	Relevant certificate
Measurement and valuation			
Right to correct certificates	No	Yes	Yes
Time frame for measurement and valuation at practical completion	Not specified	3 months	6 months if none stated
Penultimate certificate	No	Yes	Yes
Submission of documentation for final claim	Not specified	30 days of Making Good Defects Cert.	6 months after practical completion
Computation of Contract Sum	Yes	No	Yes
Final Settlement			
Final account preparation procedures	No	Yes	No
Issue of final certificate (occurrence of specified events)	No time frame specified	30 days	3 months
Effect of final certificate	Yes	Yes	No
No certificate be conclusive evidence	Yes	Yes	Yes

CONCLUSION

The results and analysis described above show that PWD 75 (the dominant and most widely used standard form of construction contract in Sarawak, despite a recent revision) is lacking in terms of completeness, as it does not cover issues such as penultimate claims, account preparation procedures and time frames for settlement and submission of final claim, whilst other standard forms, that are less widely used in the construction industry in Sarawak, such as PAM 1998 and CIDB 2000 do present viable, more complete alternatives.

Given that many items (detailed above) are not specified in PWD 75, it is argued here that this carries an inherent difficulty for contract administrators to manage the various procedural chain-of-events, which may in turn lead to imbalances of risk, differing interpretations concerning payment and certification and ultimately, dispute occurring from a lack of comprehensiveness.

Alternative forms PAM 1998 and CIDB 2000 cover procedural matters in detail adding to their comprehensiveness. However, current usage rates are significantly less than PWD 75. Given the high proportion of public sector development in Malaysia's largest state, it is perhaps logical than the PAM conditions aimed principally at the private sector, are less widely used than the Public Works Department document. Uptake of CIDB 2000 remains particularly low, with industry stakeholders somewhat intractable in their avoidance of the Construction Industry Development Board document.

Whilst academic analysis of CIDB 2000 suggests that this form provides a more appropriate balance of risk between the parties, Industry practitioners disagree and comment that it is perceived to be too contractor-friendly, offering less protection to the Client than the more traditional PWD 75. As is most often the case, Employers and their representatives decide on which form to use, and it is perhaps little wonder that after four decades of continuous and mostly unchallenged use, PWD 75 remains the standard form of choice in Sarawak, despite the shortcomings identified in this study.

Overall, this study on standard forms of contract in Malaysia presented findings that suggest that this is a merely skimming-over-the-surface exercise when it comes to understanding the underlying shortcomings in the usage of the current practice with standard forms. It is without doubt that standard forms of contract currently in use need to be reviewed and stakeholders need to recognise that:

- There are other alternatives forms available that may be better suited to available procurement processes.
- Even with the current alternatives, there may be none that represent the best solution for effective contracting, suggesting the need to further study the practice of using standard forms.
- Whilst familiarity with one particular standard forms improves the efficiency in contract administration, it does breed complacency and limits the facilitation of the necessary changes, which could bring betterment to the industry

It is important to recognise that more in depth studies with wider scope of analysis would be in line to facilitate improvements and changes necessary to establish a set of complete and comprehensive standard form that would be bring effect to Malaysian construction contracting.

REFERENCE

Ashworth, A. (1998), Civil Engineering Contractual Procedures. United Kingdom: Addison Wesley Longman Limited

Bockrath, J. T. (2000), Contracts and the Legal Environment for Engineers and Architec*ts*, 6th Ed. United States of America: McGraw-Hill Higher Education

Collier, K. (2001), Construction Contracts, 3rd Ed. New Jersey: Prentice-Hall.

Creswell, J. W. (2005), Educational research: Planning, conducting, and evaluating quantitative and qualitative research (2e). Upper Saddle River, NJ: Pearson.

Ong S. L., Tan S. I. and Oon C. K. (2000), Guide on the CIDB Standard Form of Contract for Building Work. Malaysia: Construction Industry Development Board (CIDB).

Danuri M. S., Hussain S. M. N., Mustaffa N. E. (2008), Viability of dispute avoidance procedure in the Malaysian construction industry, *Proceedings of RICS Construction and Building Research Conference, Dublin Institute of Technology* 4th -5th September 2008, available at http://www.rics.org/NR/rdonlyres/80684B62-78A1-46E5-8E64-3B2B47281DE8/0/Danuri.pdf

Rajoo, S. (1999), The Malaysian Standard Form of Building Contract (The PAM 1998 Form) (2nd Edition). Malaysia: Malayan Law Journal.

Singh, H. K. (2002). Engineering and Construction Contracts Management: Post-Commencement Practice. Malaysia: LexisNexis Business Solutions

Sia, P. S. (2008*), Evaluation on PWD 75, Standard Form of General Conditions of Contract in Sarawak*, Master Theses, Faculty of Engineering, University Malaysia Sarawak, Kota Saramahan, Sarawak, Malaysia.

Williams, K. (1992), Civil Engineering Contracts – Volume 1: The Law, Administration, Safety. England: Ellis Horwood Limited.

Vogt, W. P. (2007), Quantitative Research Methods for Professionals. Allyn & Bacon., ISBN: 0-205-35913-2

CIDB (2000), CIDB Standard Form of Contact for Building Works, Construction Industry Development Board

JKR (1961), Public Works Department (PWD 75) General Conditions of Contract, Jabatan Kerja Raya Sarawak, Government of Malaysia

PAM (1998), Agreement and Conditions of Building Contract, Pertubuhan Arkitek Malaysia MLJ 444, Lee Kam Chun v Syarikat Kukuh Maju Sdn Bhd (Syarikat Perumahan Pegawai Kerajaan Sdn Bhd, Garnishee) [1988] 1 MLJ 444).

All ER 121, P&M Kaye Ltd v Hosier & Dickinson Ltd (1972) 1 All ER 121

It was held that the words in Clause 30(7) of the JCT 1963 (which are similar to Clause 35(g) of the PWD 75) prevented any further legal action, including legal proceedings started before the certificate was issued. The certificate is not reviewable by the arbitrator under Clause 43 because it is 'conclusive evidence that any necessary effect has been given to all the terms of this Contract which require an adjustment to the Contract Sum'.

MLJ 16, Shen Yuan Pai v Dato Wee Hood Teck (1976) 1 MLJ 16.

It was found that the conclusiveness clause excuses the contractor from liability for what could be breaches of contract and obviates the operation of the Limitation Act 1953 providing that actions founded in contract and tort shall not be brought after the expiration of six (6) years from the date on which the cause of action accrued.

GUIDE TO AUTHORS

Aims and Scope:

The Malaysian Construction Research Journal (MCRJ) is the journal dedicated to the documentation of R&D achievements and technological development relevant to the construction industry within Malaysia and elsewhere in the world. It is a collation of research papers and other academic publications produced by researchers, practitioners, industrialists, academicians, and all those involved in the construction industry. The papers cover a wide spectrum encompassing building technology, materials science, information technology, environment, quality, economics and many relevant disciplines that can contribute to the enhancement of knowledge in the construction field. The MCRJ aspire to become the premier communication media amongst knowledge professionals in the construction industry and shall hopefully, breach the knowledge gap currently prevalent between and amongst the knowledge producers and the construction practitioners.

Articles submitted will be reviewed and accepted on the understanding that they have not been published elsewhere. The authors have to fill the Declaration of the Authors form and return the form via fax to the secretariat. The length of articles should be between 3,500 and 8,000 words or approximately 8 – 15 printed pages (final version). The manuscripts should be written in English. The original manuscript should be typed one sided, single-spacing, single column with font of 10 point (Times New Roman). Paper size should be of Executive (18.42 cm x 26.67 cm) with 2 cm margins on the left, right and bottom and 3 cm for the top. Authors can submit the manuscript:

- By e-mail to maria@cidb.gov.my / hazim@cidb.gov.my
- By hardcopy and softcopy in Microsoft-Word format to MCRJ Secretariat:

Malaysian Construction Research Journal (MCRJ)
Construction Research Institute of Malaysia (CREAM)
MAKMAL KERJA RAYA MALAYSIA
IBS Centre, 1st Floor, Block E, Lot 8,
Jalan Chan Sow, 55200 Kuala Lumpur
MALAYSIA

Tel. : (6)03 – 9281 0800
Fax : (6)03 – 9282 4800

Type of fonts: All text, drawing, graph must be written using Times New Roman
Language: Follow the spelling of the Oxford English Dictionary.
Size/Page Setup: Executive (18.42 cm x 26.67 cm)
Paper title: Arial, 16.

CODIFICATION AND APPLICATION OF SEMI-LOOF ELEMENTS FOR COMPLEX STRUCTURES

Author's name (full name): Arial, 9pt. should follow below the title.

Jamalodin Noorzaei[1], Mohd. Saleh Jaafar, Abdul Waleed Thanoon, Wong Jern Nee

Affiliation (including post codes): Arial, 9pt. Use numbers to indicate affiliations.

[1]Department of Civil Engineering, Faculty of Engineering, Universiti Putra Malaysia, 43400 UPM, Serdang, Selangor, Malaysia
Abstract: Arial Bold, 9pt. Left and right indent 0.25 inch.

Abstract: it should be single paragraph of about 100 – 250 words.

Keywords: Times New Roman Bold, 9pt (Italic). Left and right indent 0.25 inch.

Keywords: *Cooling tower; Finite element code; Folded plate; Semiloof shell; Semiloof beam*

Body Text: Times New Roman, 11 pt. All paragraph must be differentiate by 0.25 inch tab.

Heading 1: Arial Bold + Upper Case, 11pt.
Heading 2: Arial Bold + Lower Case, 11pt.
Heading 3: Arial Italic + Lower Case, 11pt.

Units: All units and abbreviations of dimensions should conform to SI standards.

Figures: Figures should be in box with line width 0.5pt. All illustrations and photographs must be numbered consecutively as it appears in the text and accompanied with appropriate captions below them.

Figures caption: Arial Bold + Arial, 9pt. should be written below the figures.

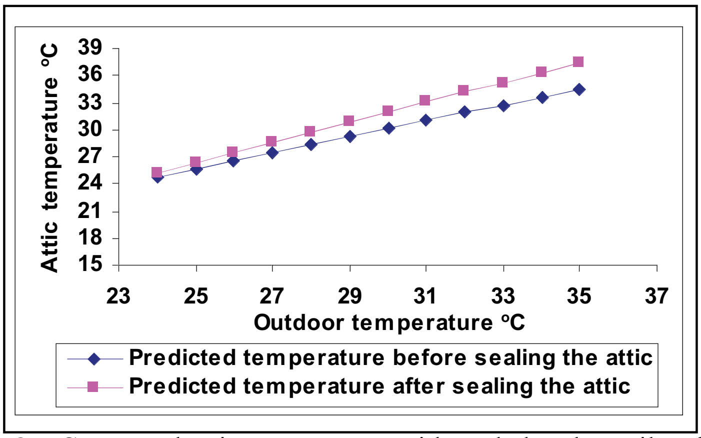


Figure 8. Computed attic temperature with sealed and ventilated attic

Tables: Arial, 8pt. Table should be incorporated in the text.

Table caption: Arial Bold + Arial, 9pt. Caption should be written above the table.

Table Line: 0.5pt.

Table 1. Recommended/Acceptable Physical water quality criteria

Parameter	Raw Water Quality	Drinking Water Quality
Total coliform (MPN/100ml)	500	0
Turbidity (NTU)	1000	5
Color (Hazen)	300	15
pH	5.5-9.0	6.5-9.0

(Source: Twort et al. 1985; MWA,1994)

Reference: Times New Roman, 11pt. Left indent 0.25inch, first line left indent – 0.25inch. Reference should be cited in the text as follows: "Berdahl and Bretz (1997) found…" or "(Bower et al. 1998)". References should be listed in alphabetical order, on separate sheets from the text. In the list of References, the titles of periodicals should be given in full, while for books should state the title, place of publication, name of publisher, and indication of edition.

Journal

Sze, K. Y. (1994) Simple Semi-Loof Element for Analyzing Folded-Plate Structures. *Journal of Engineering Mechanics*, 120(1):120-134.

Books

Skumatz, L. A. (1993) Variable rate for municipal solid waste: implementation, experience, economics and legislation. Los Angeles: Reason Foundation,157 pp.

Thesis

Wong, A. H. H. (1993) *Susceptibility to soft rot decay in copper-chrome-arsenic treated and untreated Malaysian hardwoods*. Ph.D. Thesis, University of Oxford. 341 pp.

Chapter in book

Johan, R. (1999) Fire management plan for the peat swamp forest reserve of north Selangor and Pahang. In Chin T.Y. and Havmoller, P. (eds) *Sustainable Management of Peat Swamp Forests in Peninsular Malaysia* Vol II: Impacts. Kuala Lumpur: Forestry Department Malaysia, 81-147.

Proceedings

Siti Hawa, H., Yong, C. B. and Wan Hamidon W. B. (2004) Butt joint in dry board as crack arrester. *Proceeding of 22nd Conference of ASEAN Federation of Engineering Organisation* (CAFEO 22). Myanmar, 55-64.

MCRJ Volume 6 | No. 1 | 2010

Contents

ISSN 1985-3807

9 771985 380005